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Journal of the

IRRIGATION AND DRAINAGE DIVISION

Proceedings of the American Society of Civil Engineers

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LEACHING REQUIREMENTS IN IRRIGATION

By Raymond A. Hill[1], F. ASCE

SYNOPSIS

Leaching in irrigation practice is defined and three principal purposes stated: (1) Removal of accumulated salts; (2) maintaining a salt balance; and (3) providing "equivalent service" with changes in salinity of water used for irrigation. Formulas are developed for leaching requirements related to the functions to be performed and to the salinity of applied waters.

INTRODUCTION

At the risk of over-simplification, leaching in irrigation practice may be defined as "the passage of water through soil to control salinity." The first and most commonly thought of purpose of leaching is to remove accumulated salts in soil, particularly to restore the productivity of land that has become saline. The next purpose of leaching is to maintain a "salt balance" within the root zone of irrigated land. A third, and generally unrecognized purpose, is to provide "equivalent service" with changes in the salinity of water used for irrigation.

The leaching requirement, that is, the quantity of water required to accomplish such purposes, is usually expressed as a percentage of the volume of water applied to the surface of the land, but sometimes expressed in terms of the ratio of the quantity of applied water to the consumptive use.

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[1] Pres., Leeds, Hill and Jewett, Inc., Los Angeles, Calif.

The actual requirement for leaching, however defined, depends on the function to be performed and also on the salinity of the applied waters.

REMOVAL OF ACCUMULATED SALTS

Millions of acres of land that were once productive have been abandoned because of the accumulation of salts in the soil. This was the primary cause of the collapse of ancient empires in Mesopotamia. It is the direct cause of the loss of about 100,000 acres of productive land per year in the Indus Basin. It has been the principal cause of failure of irrigation projects in the United States. It is now a major problem wherever irrigation is not accompanied by effective drainage.

Restoration of the productivity of such lands depends first on the removal of the salts that have accumulated. This can generally be done by leaching, which is the simple process of passing water through the saline soils. The significance of the words "passing through" is too frequently overlooked. The success of leaching requires that this water be carried away, either by natural drainage or through artificial drainage facilities.

Almost any kind of water can be used for leaching. Very saline waters have been found to be peculiarly effective. It is only necessary that the total amount of dissolved solids in the drainage effluent be greater than the total in the applied water. For example, even if the applied water carries 2 tons of salt per acre-ft and if the volume of the drainage effluent is only one half of the volume of the applied water, there will be a net removal of salt from the soil so long as the concentration of salts in the drainage effluent is more than 4 tons per acre-ft, roughly 3,000 parts per million. In actual practice most applied waters will contain less than 2 tons of dissolved solids per acre-ft and most drainage effluent during this type of leaching operation will have concentrations of salts of from 5 to 10 tons per acre-ft.

However, the leaching requirement for this purpose cannot be expressed as a percentage of the applied water because the ratio of the concentration of salts in the drainage effluent to the concentration in the applied water will diminish progressively during the leaching period. This ratio might start out at 5:1 and gradually reduce to little more than 1:1 as the soluble salts in the soil are picked up and carried away by the waters percolating downward.

SALT BALANCE

After land has been restored to productivity by the removal of excess salt, it is necessary to maintain a salt balance within the root zone. The same is true with respect to new lands; a salt balance must be maintained at all times. Continued productivity of land depends on the prevention of any accumulation of salts having an adverse effect on crops.

Maintenance of a salt balance merely means that the quantity of salt carried away in the percolate below the root zone should be equal to the quantity of the same salt in the water applied to the surface of the land. Generally, it is deemed sufficient if the total of tons of salt carried away from the land is as great as the total of tons of salt delivered to the land in the water applied. This is not strictly true, because some ions precipitate out forming insoluble compounds, and base exchange frequently takes place. The generalization, however, is sufficiently accurate for the purposes of this paper.

By definition, the volume of the percolate (P) equals the difference between the quantity of applied water (D_a) and the consumptive use (D_o). Maintenance of a salt balance would require only that

$$D_a C_a = P C_p \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

and

$$\frac{P}{D_a} = \frac{C_a}{C_p} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which duty of water D_a refers to the quantity of water, in depth per unit area, applied to the land; P denotes the amount of percolate from root zone, in depth per unit area; C_a is the concentration of irrigation water applied to the land, either in respect to total dissolved solids or to any deleterious constituent; and the concentration of percolate C_p, denotes the concentration of the water which passes out of the root zone. For example, if there were 1 ton of salt per acre-ft in the applied water and 4 tons per acre-ft in the percolate, the leaching requirement would be one-fourth, or 25%, of the volume of the applied water. In like manner, if the concentration of the applied water were 2 tons per acre-ft and that of the percolate were 8 tons per acre-ft, the ratio would be the same and the leaching requirement would still be 25% of the volume of the applied water.

If, in the basic equation, the expression $(D_a - D_o)$ were substituted for P

and

$$D_a C_a = D_a C_p - D_o C_p \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

$$\frac{D_a}{D_o} = \frac{C_p}{C_p - C_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

in which the consumptive use D_o refers to the minimum duty of water, in depth per unit area, required only to support crop growth and meet evaporation losses. Substitution of either set of values from the preceding examples gives 4:3 as the ratio of the volume of applied water to the consumptive use, which corresponds to a volume of percolate equal to 25% of the quantity of applied water or to one third of the consumptive use.

EQUIVALENT SERVICE

In each of the foregoing examples, a salt balance is maintained by application of one-third more water than that required for consumptive use irrespective of the concentration of salts in the applied water. The question then arises as to whether maintenance of a salt balance is a true and sufficient measure of the leaching requirement using saline waters. This question was raised in about 1937, in connection with irrigation projects along the Rio Grande River in Colorado, New Mexico, and Texas, but the answer then given has not received wide acceptance.

Let us consider two divisions of an irrigation project along a river. The upper division receives water having a salt concentration of 1.0 ton per acre-ft. The supply of the other is a mixture of water from the same source and drainage return from the upper division, and this mixture contains 2.0 tons

of salt per acre-ft. If it is then assumed that the percolate from irrigated fields in the upper division contains 5.0 tons of salt per acre-ft, the ratio of the quantity of applied water to the quantity of percolate would be 5:1. If it is further assumed that the consumptive use is 3.0 acre-ft per acre, the farm duty would be 3.75 acre-ft per acre in the upper division of this project. If the same farm duty and consumptive use be used for the lower division, the percolate from irrigated fields in that division would have a salt concentration of 10.0 tons per acre-ft. Under these conditions, the duty of water would be the same in both divisions, the consumptive use would be the same, and a salt balance would be maintained in each division. Yet, can it be said that the lands in the lower division of this project are being given service equivalent to that given those in the upper division? Obviously not, unless the salinity of an irrigation supply were of no significance.

In 1937, after consultation with C. S. Scofield and others connected with the United States Department of Agriculture, Rubidoux Salinity Laboratory at Riverside, California, the following principle was developed:

> "Whenever the quality of water applied to land for irrigation is modified by an increase or decrease in the concentration of one or more ions detrimental to plant growth, maintenance of the same effective concentration of the soil solution, by increasing or decreasing the amount of water applied, will constitute equivalent service for the growing of crops."

The concentration of the soil solution naturally varies from the surface to the lower limit of the root zone and likewise varies depending on the time since irrigation. The minimum average concentration of the soil solutions at all horizons in the root zone shortly after irrigation should be about equal to the arithmetic average of the concentration of the irrigation water and of the percolate from the root zone. This average has been taken as an index of the effective concentration (C_S) of the soil solution.

The effective concentration of soil solution, C_S, is considered to be the arithmetic average of the concentrations of the irrigation water applied and the percolate from the root zone, thus

$$C_S = \frac{C_a + C_p}{2} \quad \ldots\ldots\ldots\ldots\ldots \quad (5)$$

or

$$C_p = 2\,C_S - C_a \quad \ldots\ldots\ldots\ldots\ldots \quad (6)$$

However, for salt balance

$$D_a = D_o \left(\frac{C_p}{C_p - C_a} \right) \quad \ldots\ldots\ldots\ldots\ldots \quad (7a)$$

Substituting Eq. 6 into Eq. 7 yields

$$D_a = D_o \left(\frac{2\,C_S - C_a}{2\,C_S - 2\,C_a} \right) \quad \ldots\ldots\ldots\ldots\ldots \quad (7b)$$

Using the values in the foregoing example, the effective concentration of the soil solution in the upper division is 3.0 tons per acre-ft and the farm duty remains 3.75 acre-ft per acre. This gives a leaching requirement equal to 20% of the volume of applied water. In the lower division, however, maintenance of the same effective concentration of the soil solution, C_s = 3.0 tons per acre-ft, requires(from Eq. 7b) that the farm duty be 6.0 acre-ft per acre: (C_a =2.0 tons per acre-ft D_o = 3.0 acre-ft per acre)

$$D_a = 3.0\left(\frac{6.0 - 2.0}{6.0 - 4.0}\right) = 6.0$$

The percolate, which is the difference between the volume of applied water and the consumptive use, would thus be 3.0 acre-ft per acre, corresponding to a leaching requirement of 50% of the volume of the applied water. The concentration of the percolate would then be only twice the concentration of the applied water or 4.0 tons per acre-ft. This, of course, gives 3.0 tons as the effective concentration of the soil solution, taken as the arithmetic average of the concentrations of the applied water and the percolate.

It follows, if the salinity of an irrigation supply is significant, that it is not enough only to provide for maintenance of a salt balance. The concentration of salts in the percolate must be diminished as the concentration of salts in the applied water increases. Otherwise, there will not be equivalent service for the growing of crops.

CONCLUSIONS

One of the primary purposes of leaching, and the only one generally recognized by laymen, is the removal of prior accumulations of salt to restore the productivity of land. This requires that large volumes of water be applied on the land surface; that the water be caused to percolate through the soil; and finally, that the percolate, which has picked up the accumulate salt, be disposed of as drainage effluent.

The second and always important function of leaching is the maintenance of a salt balance in the root zone of irrigated lands. This requires the passage of enough water through the soil to carry away the salts that are brought to the land in the water applied for irrigation.

The third function of leaching, within the basic definition, is to control the salinity of the soil solution from which the plants derive water. This function is not recognized generally, but the leaching requirements is necessarily greater where strongly saline waters are used than where moderately saline waters are available for irrigation of crops.

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INTERCEPTING DRAINAGE WELLS IN ARTESIAN AQUIFER

By Dean F. Peterson, Jr.,[1] F. ASCE

SYNOPSIS

A formula for channel seepage to a leaky artesian aquifer and for the resulting piezometric surface is presented. A potential function accounting for excess irrigation and leaching water applied to agricultural lands is proposed. Using this function in combination with conventional well potential theory, a method for designing intercepting drainage wells for such an aquifer underlying irrigated lands is described.

INTRODUCTION

Occasionally drainage problems arise on low lying lands overlying artesian aquifers fed by streams at higher elevation. This case may occur on the sea deltas of large rivers. The Sacramento-San Joaquin Delta in California is typical. Here some half million acres lie at elevations generally below sea level. This area consists of more than 50 islands interlaced with tidal channels. The soils are organic or mineral organic, generally 20 ft to 30 ft thick, overlying mineral sands into which the tidal channels penetrate and which act as aquifers replenished by the water in the tidal channels. The penetration of saline waters into the tidal channels as the fresh river supply is depleted by

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[1] Prof. Civ. Engrg. and Dean, College of Engineering, Utah State Univ., Logan, Utah.

irrigation may endanger the agricultural lands overlying the aquifer. The purpose of this paper is to describe methods of analysis leading to the design of well interceptors to control the artesian water.

LEAKAGE FROM THE AQUIFER

An idealized representation of the condition previously described is shown in Fig. 1. The excess artesian head at the edge of the region is denoted by h_o and h_x is the same at a horizontal distance x. The terms q_o and q_x are the horizontal flow per unit width in the aquifer at x = o and x, respectively. The effective upward hydraulic conductivity of the overlying soil is denoted by K_v and T is the transmissivity of the aquifer. The following equations may be written from Darcy's law:

$$\frac{dq_x}{dx} = K_v \frac{h_x}{b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

and

$$q_x = - \frac{dh_x}{dx} T \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Integrating Eq. 1 yields

$$q_x = q_o - \int_o^x K_v \frac{h_x}{b} dx \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Substituting Eq. 2 in Eq. 3 yields

$$- \frac{dh_x}{dx} T = q_o - \int_o^x K_v \frac{h_x}{b} dx \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

which, differentiated with regard to x, leads to

$$T \frac{d^2h_x}{dx^2} = \frac{K_v}{b} h_x \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

An appropriate solution for Eq. 5 is

$$h_x = h_o e^{- \sqrt{K_v/(b\,T)}\, x} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

If x becomes large, in Eq. 3 $q_x \rightarrow o$.
Thus

$$q_o = \int_o^\infty \frac{K_v}{b} h_x dx = \frac{K_v}{b} \int_o^\infty h_o e^{- \sqrt{K_v/(b\,T)}\, x} \quad \ldots\ldots (7a)$$

and

$$q_o = \sqrt{\frac{K_v\, T}{b}}\; h_o \quad \ldots\ldots\ldots\ldots\ldots\ldots (7b)$$

Eqs. 6 and 7b may be used to estimate the drawdown curve and the leakage, q_o, if K_v, T, b, and h_o are known. They may also be used as analytical devices to estimate K_v and T from field investigations. These investigations might include hydrologic balance studies leading to estimation of the unit channel loss q_o and measurements of the piezemetric surface for the artesian aquifer.

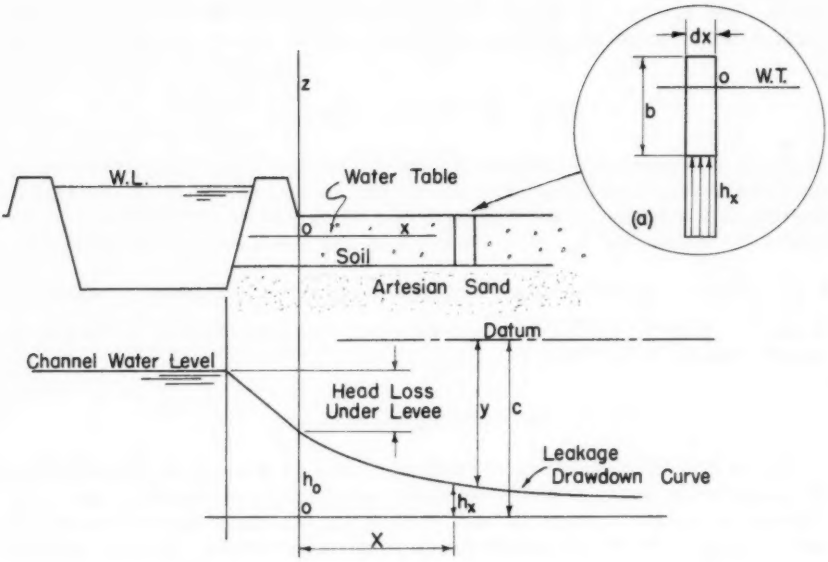

FIG. 1.—DEFINITION SKETCH

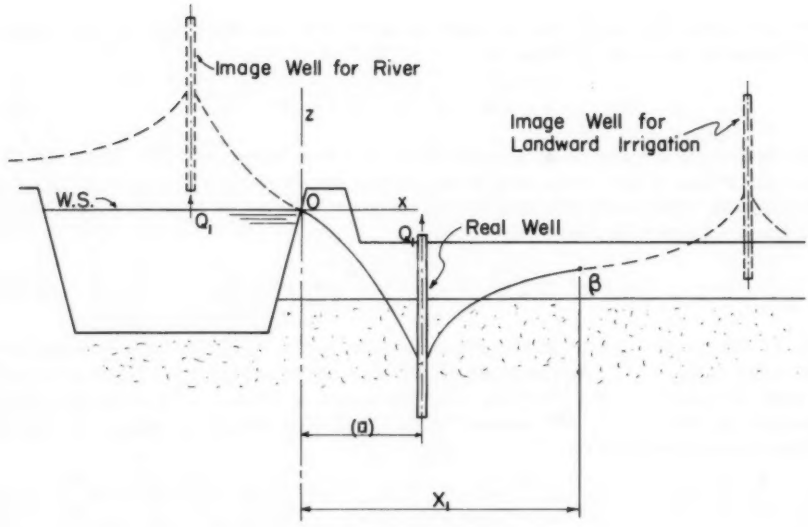

FIG. 2.—IMAGE WELLS REPRESENTING CHANNEL SUPPLY AND LANDWARD IRRIGATION

An appropriate value of h_o may be induced from the drawdown measurements. Let $h_x = z\ c$, in which z is the measured elevation of the piezometric surface with reference to some arbitrary datum, Eq. 6 may be rewritten,

$$\text{Log}\,(z - c) = \log h_o - \frac{1}{2.3}\sqrt{\frac{K_v T}{b}}\ x \quad \ldots\ldots\ldots\ldots (8)$$

In Eq. 8, z is positive if measured upward from the datum, negative if measured downward. Assuming successive trial values of c, log (z - c) may be plotted as a function of x. The value of c which yields a straight line establishes the datum for h_x. The intercept of this line is log h_o and its slope is $-\frac{1}{2.3}\sqrt{\frac{K_v T}{b}}$. Substituting these values in the preceding gives an equation in K and T. Knowing q_o, Eq. 7b provides a simultaneous equation which may be solved with Eq. 8 for T and K.

INTERCEPTING WELLS

Use of Potential Functions.—Potential functions provide an excellent way to combine the various factors involved into a single expression. For engineering purposes a potential function Z having the dimensions of energy per unit of weight (FL/F) or simply length is most convenient. For the real well at x = a and y = b, Z is the elevation of the piezometric surface.

$$Z_1 = \frac{2.3\ Q_1}{4\ \pi\ T}\ \log\left[(x - a)^2 + (y - b)^2\right] \quad \ldots\ldots\ldots\ldots (9)$$

For the image recharge well at x = -a and y = b, the discharge is -Q. Thus the potential function is (Fig. 2)

$$Z_1 = -\frac{2.3\ Q_1}{4\ \pi\ T}\ \log\left[(x - a)^2 + (y - b)^2\right] \quad \ldots\ldots\ldots\ldots (10)$$

The boundary requirement that Z = 0 for x = 0 is assured by the image well, thus the effect of the river may be accounted for by adding the potential function for an imaginary recharge well mirrored about the boundary axis, thus for the well and river system,

$$Z_1 = \frac{2.3\ Q}{4\ \pi\ T}\ \log\ \frac{(x - a)^2 + (y - b)^2}{(x + a)^2 + (y - b)^2} \quad \ldots\ldots\ldots\ldots (11)$$

Eq. 11 becomes infinite at the centers of the wells, but may be computed for the edge of the well. Any number of wells at other particular values of a and b may be added to the system. For example, a system of 3 wells in a line parallel to the river, and spaced at 2 a with the origin as shown in Fig. 3 would be represented by

$$Z_1 = \frac{2.3\ Q}{4\ \pi\ T}\ \log\left[\frac{(x - a)^2 + y^2}{(x + a)^2 + y^2}\right]\left[\frac{(x - a)^2 + (y - 2\,a)^2}{(x + a)^2 + (y - 2\,a)^2}\right]\left[\frac{(x - a)^2 + (y + 2\,a)^2}{(x + a)^2 + (y + 2\,a)^2}\right] \quad \ldots (12)$$

Irrigation Function.—If the agricultural land is provided with an excess irrigation equal to q_v (volume per unit area per unit time) due to deep per-

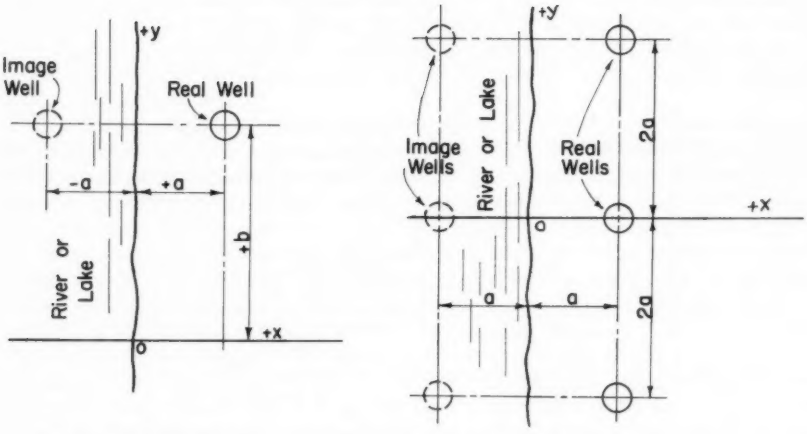

FIG. 3.—PLAN ARRANGEMENT OF WELLS

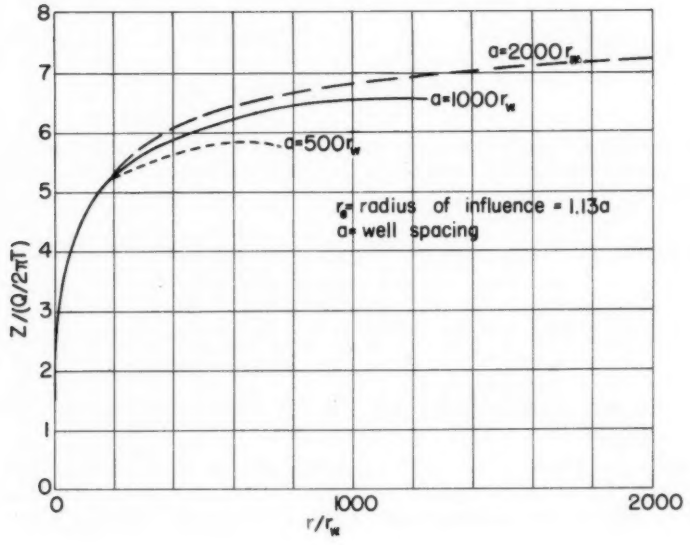

FIG. 4.—POTENTIAL FUNCTION FOR EXCESS IRRIGATION IN INFLUENCE ZONE

colation losses and for leaching purposes, the piezometric effect within a well influence zone is[2]

$$Z_2 = \frac{Q_2}{2\pi T}\left[2.3 \log \frac{r}{r_w} - \frac{1}{2}\left(\frac{r}{r_e}\right)^2\right] \quad \ldots\ldots\ldots\ldots (13)$$

in which

$$Q_2 = \pi r_e^2 q_v \quad \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

in which r_w is the effective radius of the well, r_e is the radius of the influence zone of a particular well and r, the radial distance from the well to the point

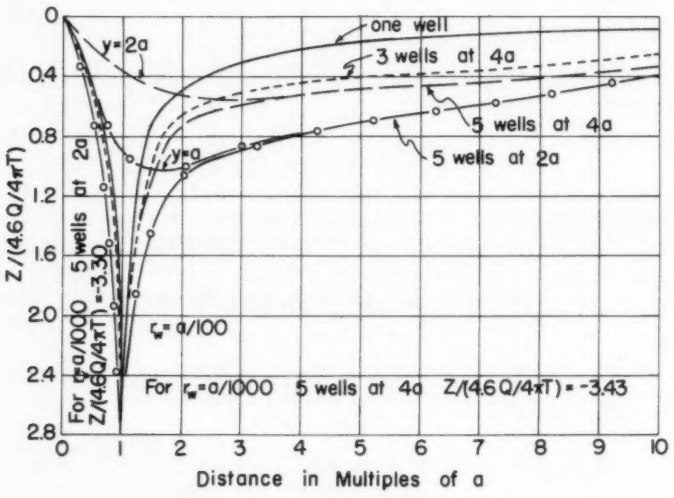

FIG. 5.—POTENTIAL FUNCTIONS FOR VARIOUS ARRANGEMENTS OF WELLS IN A SINGLE LINE PARALLEL TO THE CHANNEL

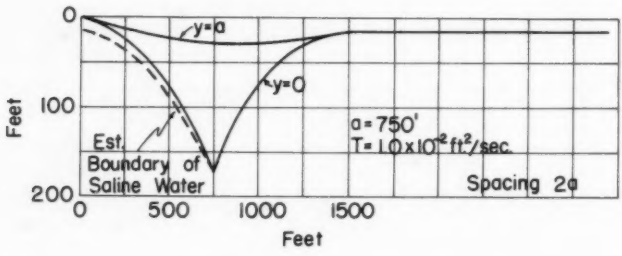

FIG. 6.—DRAWDOWN CURVES FOR 5 WELLS IN A SINGLE LINE

where Z is desired. The excess irrigation water intercepted by the well is denoted by Q. The origin for Eq. 14 is at the piezometric level in the well. Graphs of the irrigation function are shown in Fig. 4. Since drainage wells

[2] "Hydraulics of Wells," by Dean F. Peterson, Transactions, ASCE, Vol. 122, 1957.

are usually spaced at the corners of squares, the value of r_e is taken as the radius of a circle having the same area as a square with sides d equal to well spacing, or, $r_e = 1.13\ d$. While the potential functions of Eqs. 9, 10, and 11 are superposed throughout the entire region, the function expressed by Eq. 13 terminates at the boundary of the influence zone for the particular well with which it is associated.

Landward Irrigation.—Quantitative values of Z_1 and Z_2 are not defined by the choice of a and b, q_v and the elevation of the river water surface. Each value of drawdown is associated with some value of total discharge Q. If one chooses to make some decision about the piezometric surface to be maintained at some point β, distance x from the river, sufficient boundary values are introduced to provide a solution. The elevation of point β may be maintained by a set of image wells about that point as shown in Fig. 2. By combining these image wells with the real wells, a function Z_3 associated with the landward irrigation is obtained. This function is identical with Z_1 rotated 180° about the real well and having a transformed horizontal scale of $(x_1 - a)/a$.

Example.—Fig. 5 shows dimensionless values for Z for several different numbers and arrangements of wells in a single line at distance a from the river in a cross section perpendicular to the river through the central well. In order to illustrate the use of the proposed method of analysis, assume an array of 5 wells in line at 750 ft from the river and spaced 1,500 ft apart. Assume q_v is 2.5 ft per yr $(8.0 \times 10^{-8}$ ft per sec), T is 1.0×10^{-2} ft per sec, and the piezometric surface at β, 1,500 ft from the river is to be maintained at -18 ft. The radii of the wells are 0.75 ft.

$$Q_2 = (1.13 \times 750)^2 \left(8 \times 10^{-8}\right) \pi = 0.181 \text{ cfs}$$

For r = 750 ft, $r/r_w = 1{,}000$, from Fig. 3, $\dfrac{Z}{\dfrac{Q}{2\pi T}} = 6.5$;

$$Z_2 \text{ at } r = 750 \text{ ft} = \frac{(6.5)\ (.18)}{2\ \pi\left(10^{-2}\right)} = 18.6 \text{ ft}$$

Values of Z_2 at other values of r may be obtained by multiplying the values from Fig. 4 by 18.6/6.5 or 2.86. The value of Z_3 is determined by the boundary value at x = 0 in which $Z_1 + Z_2 + Z_3 = Z = 0 + 18.6 + Z_3 = 0$; $Z_3 = -18.6$ ft.

From Fig. 5 (rotated 180°) $\dfrac{Z_3}{\dfrac{4.6\ Q}{4\ \pi\ T}} = -1.06$ and other values of Z_3 may be found by multiplying the values of $\dfrac{Z_3}{\dfrac{4.6\ Q}{4\ \pi\ T}}$ by 18.6/1.06 or 17.6 Similarly Z_1 is determined by the boundary condition at β in which, from Fig. 5, $\dfrac{Z_1}{\dfrac{4.6\ Q}{4\ \pi\ T}} = -1.06$.

Here $Z = Z_1 + Z_2 + Z_3 = Z_1 + 18.6 + 0 = -18$ ft; $Z_1 = -36.6$ ft and the other values of Z_1 may be computed by multiplying the values from Fig. 5 by 36.6/1.06 = 34.5.

$$Q_3 = \frac{4\ \pi\ T\ Z_3}{(1.06)\ (4.6)} = \frac{4\ \pi\left(10^{-2}\right)(18.6)}{(1.06)\ (4.6)} = 0.48 \text{ cfs}$$

and

$$Q_1 = \frac{4\,\pi\,(10^{-2})\,(36.6)}{(1.06)\,(4.6)} = 0.95 \text{ cfs}$$

The total discharge Q is 1.61 cfs. At the well, $Z/\left(\frac{4.6\,Q}{4\,\pi\,T}\right) = 3.30$, and the drawdown may be computed to be, remembering $Z_2 = 0$, $D = Z_1 + Z_2 + Z_3 = 0 - 3.30\,(17.6 + 34.5) = 171$ ft. Fig. 6 shows sections through the resulting piezometric surface.

CONCLUSIONS

A simplified expression for the seepage from a channel to a leaky artesian aquifer is $q_o = \frac{\sqrt{K_v\,T}}{b}\,h_o$ in which K_v is the effective vertical hydraulic conductivity and b is the thickness of the overlying layer, T is the transmissivity of the aquifer and h_o is the excess piezometric head in the aquifer at the edge of the region. The resulting drawdown curve is given by $h_x = h_o\,e^{-\sqrt{K_v/(T\,b)}\,x}$ in which h_x is the excess piezometric head at distance x from the channel.

Introducing a potential function describing the excess irrigation and leaching water applied in the well influence zone and using the method of images to describe the effect of excess irrigation and leaching water applied outside of the influence zone leads to a rational method using potential theory for designing drainage wells intercepting such an artesian aquifer. The methods used for this particular case, especially the use of the irrigation function, may be extended to other cases of drainage well design.

ACKNOWLEDGMENTS

This paper is based on methods developed in connection with a study made by the writer for Wynne Thorne Associates, agricultural consultants to Charles T. Main, Inc. of certain aspects of drainage relating to the Sacramento-San Joaquin Delta.

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WATER BALANCE RECORDER

By Hans C. Aslyng[1] and Knud J. Kristensen[2]

SYNOPSIS

This paper presents a detailed description of the design and construction of a semi-floating lysimeter. The installation consists of two tanks, one inside the other. The inner tank is partly floating and partly counterbalanced. Based on change in weight, the change in water level between tanks is amplified and recorded on a clock-driven chart. Results, based on the use of such a recorder, are presented.

INTRODUCTION

Lysimeters of various designs have been used for the study of water balance. Many of the results obtained are not valid for crops grown under natural conditions as a result of border effects. It is necessary that the lysimeter be indistinguishable from a large surrounding natural area.

In the study of precipitation, evaporation, and evapotranspiration, it is desirable to be able to record accurately and continuously. For that reason weighable lysimeters or evapotranspirometers are used, and different types are de-

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scribed in the literature. Some are manually operated[3] and others have mechanisms which automatically record their weight or change in weight.[4,5,6,7,8,9]

Two distinctly different methods are applied: Recording by balances of different types as used by Arthur,[4] Morris,[7] Harrold and Dreibelbis,[6] and Pruitt and Angus,[9] or by the Archimedean principle as used by King[5] and Popov.[8]

A combination of the two methods is used for the water balance recorder to be described (Fig. 1). Two such recorders are placed 20 km (65,617 ft or 12.5 miles) west of Copenhagen, Denmark at the Climate and Water Balance Station of the Hydrotechnical Laboratory.

PRINCIPLE

The installation consists of two tanks, one within the other, with 1 cm (0.3937 in.) space between them. Between the two tanks water fills the annular space to about 35 cm below the upper rim. This water carries part of the tank weight. The inner tank is fixed in the center of the outer one by means of the lever on which it rests. The other end of the lever carries a weight which counterbalances the rest of the tank weight (Fig. 1). The counterbalancing system does not effect the weighing process.

Change in the weight of the inner tank influences the recorded water level between the tanks. The recording system consists of a small water tank connected to the water between the two tanks by means of iron pipes. A float in the water tank operates a lever with a pen attached which records the change in weight on a chart which is mounted on a clock drum.

CONSTRUCTION

The inner tank has an inside diameter of 159.6 cm, giving an area of 2.0 sq m (21.53 aq ft). The depth is 90 cm to 105 cm and the total volume about 2 cu m (70.63 cu ft).

An iron bar, 180 cm long, 3 cm wide and 10 cm high, is mounted across the inner tank so that the lower side is 25 cm below the top rim. The ends of this bar have knife edges which rest on a lever bent around the outer tank. On each side of the outer tank, a piece 28 cm by 5 cm is removed to make space for the bar (Fig. 2).

The bottom of the inner tank is an iron plate 1 cm thick that is curved downwards to increase the strength. In order to secure sufficient strength and for

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3 "Een nieuw lysimeterstation," by G. F. Makkink, Water 13, 1953, pp. 159-163.

4 "An Evapotranspirometer," by I. P. Arthur, Australian Journal of Agric. Research, No. 6, 1955, pp. 707-712.

5 "A Floating Lysimeter and its Evaporation Recorder," by K. M. King, C. B. Tanner, and V. E. Suomi, Transactions, American Geophysical Union, No. 37, 1956, pp. 738-742.

6 "Evaluation of Agricultural Hydrology by Monolith Lysimeters," by L. L. Harrold and F. R. Dreibelbis, Technical Bulletin 1179, U. S. Dept. of Agric., 1958, 166 pp.

7 "A Recording Weighing Machine for the Measurement of Evapotraspiration and Dewfall," by L. G. Morris, Journal of Agricultural Engineering Research, No. 4, 1959, pp. 161-173.

8 "Lysimeters and Hydraulic Soil Evapcrimeters," by O. V. Popov, Publication No. 49 of the Assn. of Internatl. Hydrologic Science Publications, 1959, pp. 26-37.

9 "The Davis Weighing Lysimeter," by W. O. Pruitt and D. E. Angus, from "An Introduction to Physical Micro-Climatology," by F. A. Brooks, Univ. of California, 1959, pp. 129-132.

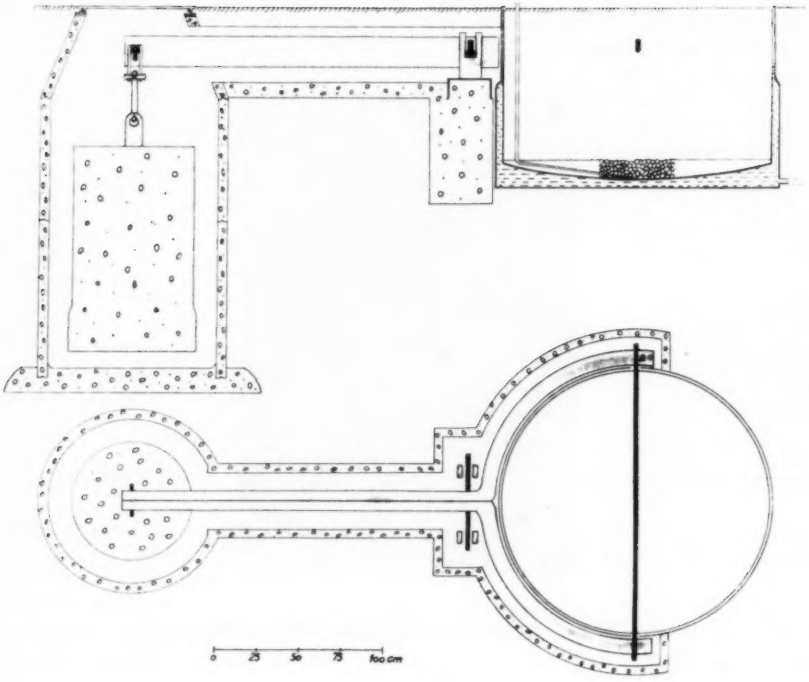

FIG. 1.—VERTICAL AND HORIZONTAL CUTS OF THE FIELD PART OF THE WATER BALANCE RECORDER

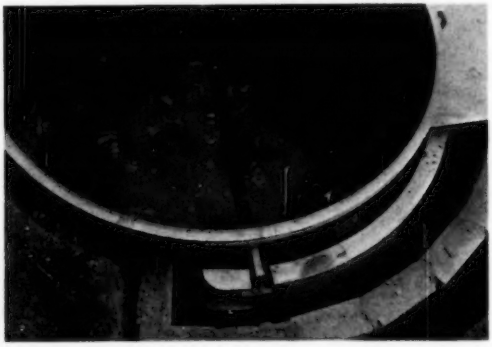

FIG. 2.—ONE SIDE OF THE TANKS AND THE BAR WITH THE KNIFE RESTING ON THE LEVER

stability in shape, an iron plate 2 cm thick and 10 cm wide is welded to each side of the tank from the bottom to the knives. The wall of the tank is made of a 0.5 cm thick iron plate.

FIG. 3.—THE LEVER AT THE COUNTERWEIGHT

FIG. 4.—THE LEVER AT THE CONCRETE BLOCK AND TANKS

The inner tank has a false bottom at a depth of 90 cm. It consists of a 0.5 cm thick iron plate, the center of which is cut away leaving a circular opening with 45 cm diameter in the middle of the tank. Around this opening the bottom of the tank is supported by a perforated, 15 cm high iron cylinder. The cham-

ber between the two bottoms can contain about 120 l (approximately 306 gal) of water. A 1.2-cm-wide iron pipe reaches from the lowest part of the tank to the surface. Water can move in either direction through this pipe. The center compartment at the bottom of the tank is filled with stones (1 cm to 3 cm in diameter). On the false bottom and on the stones in the center compartment is placed a 3 cm-to-4-cm thick layer of gravel to facilitate water movement. The tank is filled with 60 cm of underground sandy clay loam and 30 cm of sandy clay loam of topsoil with about 3% organic matter. It is thus filled with the same material as the surrounding soil. Investigations can be carried out with or without the soil in contact with "ground water."

The total weight of the filled inner tank is 3700 to 3800 kg (approximately 8170 lb to 8400 lb). About one-third is "floating" and about two-thirds of the weight is counterbalanced.

The lever is made from two U-shaped iron girders which, when welded together, form an I-shaped iron bar. The lever has a ratio of 1:2 for tank-counterweight. All knife edges and the center of gravity of the bar are in the same plane to avoid the possibility of the lever influencing the weighing process. The bent parts and the bearing points of the lever are strengthened by welding on 2 cm-by-15.5 cm iron plates (Figs. 2, 3, and 4).

The inner tank, the counterweight, and the lever are carried by a 40 cm-by-100 cm concrete block. The total load amounts to about 1 kg per sq cm (14.2234 psi).

An underground canal covered by iron plates (0.5 cm thick) and made waterproof by sheets of plastic, is constructed around the lever. The canal is covered by a thin layer of soil.

The counterweight is made of concrete. It is cylindrical with a diameter of 72 cm and a length of 125 cm. Its total weight of about 1250 kg can be increased if necessary. The counterweight is suspended in a sufficiently deep waterproof well.

The outer tank is made of a 0.5 cm thick iron plate. Its upper, inside diameter is 162.6 cm. At a point 40 cm below the rim, the diameter is increased by 7 cm to make more space for movement of the inner tank.

Four "stoppers" are placed at the bottom of the outer tank. These limit the downward movement of the inner tank to a maximum of 2 cm. The inner tank can rest on these stoppers during the winter at which time the water between the tanks is removed.

The water level in the 1 cm space between the two tanks is about 35 cm below the upper rim and is covered by a 1 cm transformer oil (univolt 35) to avoid evaporation.

At ground level, the space between the two tanks is covered by a 3.5-cm-wide iron ring. The ring rests on the outer tank and reaches 0.5 cm inside the inner tank, the upper rim of which is 1 cm to 2 cm below the upper rim of the outer tank. A thin iron band (1.5 cm-by 0.1 cm) is welded on the inner edge of the ring, so that the open tank area is exactly defined, and so that in the downward direction, it forms a "nose" which prevents water from dripping into the space between the tanks.

All iron parts are galvanized and surfaces in contact with the soil are asphalt coated.

The water connection between the two tanks is made by means of galvanized iron pipes (2.5 cm inside diameter) and a stiff rubber hose connected to the float tank of the registration installation. The latter is placed in a small, partly underground building about 20 m (65.6 ft) away.

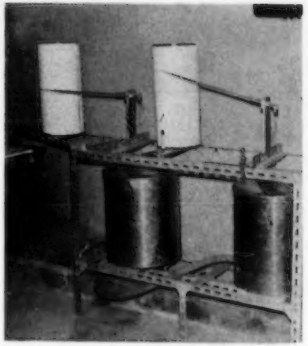

FIG. 5.—THE RECORDING SYSTEM

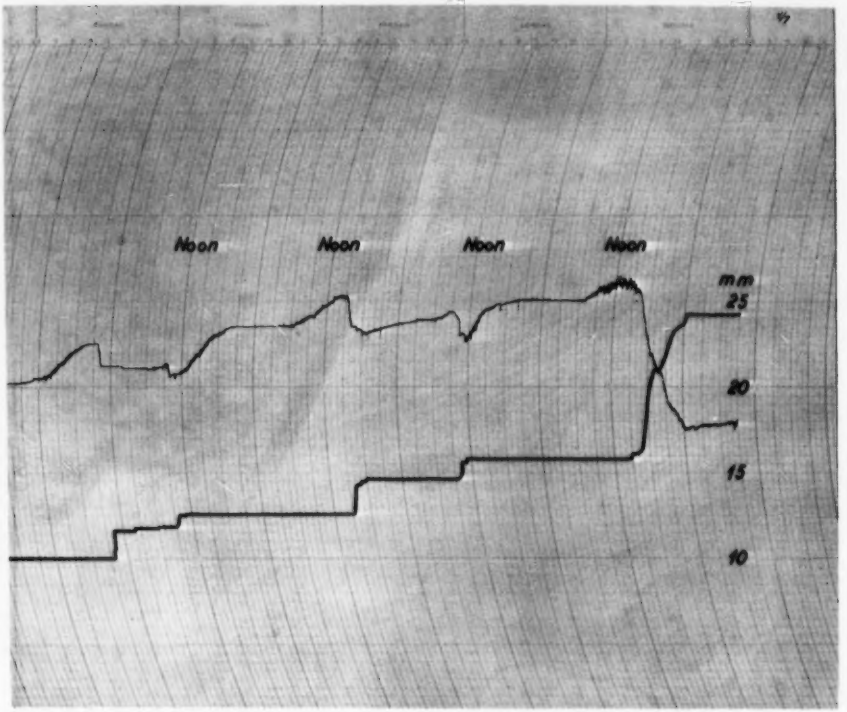

FIG. 6.—UPPER CURVE: THE RECORDED CHANGE IN WATER BALANCE DUE TO RAINFALL AND DUE TO EVAPORATION FROM BARE SOIL DURING A PERIOD OF FOUR DAYS. LOWER CURVE: RECORDED RAINFALL IN THE SAME PERIOD. 1 mm WATER CORRESPONDS TO 1 cm ON THE CHART

The float tank and the float are made of 0.1 cm brass plate. The tank has an inside diameter of 35 cm and a height of 50 cm. The float is partly built as a cone. Its outside diameter is 33 cm with a maximum height of 23 cm. In order to give it strength so as to operate the pen securely, the mass of the float is increased by using a ballast of lead. The water surface in the float tank is covered by 1 cm of transformer oil in order to avoid evaporation. The float is maintained by means of a flexible wire connected to a pen lever with knives and ball bearings. The lever, 55 cm long, consists of a 45 cm light U-shaped brass bar and a 10-cm-long pen holder made of a 0.1 cm brass plate. The pen holder can swing in the horizontal plane and it is mounted in such a way that the pen exerts a light pressure on the chart (Fig. 5).

The drum for the chart is made of a fibrous material of low density. In the drum is an ordinary clock which makes a rotation per week. The drum is 43 cm high and it is 63 cm in circumference. Each centimeter on the chart represents 1 mm (0.0394 in.) precipitation or evaporation. The total scale on the chart corresponds to 40 mm of water.

ANALYSIS

As mentioned previously it is important that the lysimeter be indistinguishable from the surrounding area. Thus the rim of the completed system should be as narrow as possible and level with the surrounding ground.

In this installation, the total width of the rim is 3.5 cm giving a border area of 1800 sq cm or 9% of the tank area. This rather narrow rim will be almost invisible when the tank and its surroundings are cropped.

The soil surface level in the tank changes by only approximately 0.3 cm for 40 mm precipitation or evaporation. The small variation in level will have no significant effect on the water balance in the tank.

The ability of the installation to record changes in weight of the tank appears from the upper curve of Fig. 6. Increases in the recorded curve are due to evaporation. Thus decreases are caused by weight. Rainfall causes a decrease in the curve. The ordinate units are millimeters of water. Each millimeter of water equals 2 kg. The results are obtained with bare soil in the tank, but will be similar with cropped soil.

The lower curve of Fig. 6 shows the recorded rainfall for the same period. It can be seen that the water balance recorder records rainfall in agreement with the rainfall recorder. The accuracy of the water balance recorder is of the order of 200 g or 0.1 mm of water. That is about the smallest quantity that can be distinguished on the chart. The field installation might be sensitive to even smaller quantities. It is expected that information concerning dewfall also can be obtained by the water balance recorder.

The rather great periodical oscillation in the upper curve is due to gusty winds. This has been eliminated by increased resistance to water flow in the connecting pipe line.

The advantages of the water balance recorder are that the soil depth in the tank is large and uniform because air chambers are avoided; and that the tank is fixed in the center of the outer tank. The presence of water between the two tanks is of importance for thermal equalization between the tanks and the surrounding soil. The simple and cheap recording system is also an advantage.

By identical water balance recorders it is possible to make comparisons concerning the effect on the water balance due to different soil tillage or cropping systems.

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RELATION OF INTAKE RATE TO LENGTH OF RUN IN SURFACE IRRIGATION

By A. Alvin Bishop,[1] F. ASCE

SYNOPSIS

The relationship between the intake rate of the soil and the length of run in surface irrigation with regard to the amount of water lost below the root zone through deep percolation is presented. A nomograph is included from which the percentage of loss by deep percolation can be estimated for soils having different intake characteristics.

Low application efficiencies in surface methods of irrigation have generally been charged to deep percolation losses below the root zone. Excessive deep percolation near the head ditch or upper end of the run to allow the lower end to become fully irrigated is generally assumed to be the major source of surface irrigation losses. The fact that water losses are generally high was shown by O. W. Israelsen, F. ASCE et al.[2] in reporting the results of application efficiency measurements on a number of Utah farms. Thirty-nine tests on one group of farms showed an average efficiency of application to be 38% whereas 90 tests on a second group of farms showed the efficiency to be 44%. Similar results have been demonstrated by tests in other areas.

Note.—Discussion open until August 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. IR 1, March, 1961.

[1] Prof., Civ. and Irrig. Engrg., Utah State Univ., Logan, Utah.

[2] "Water Application Efficiencies in Irrigation," by O. W. Israelson, Wayne D. Criddle, Dean K. Fuhriman, and Vaughn E. Hansen, Bulletin No. 311, Utah Agric. Experiment Sta., March, 1944.

A mathematical analysis of the deep percolation loss was presented by Wayne D. Criddle, M. ASCE et al.[3] for a particular length of run for a soil having given intake characteristics. In the case analyzed, the water was assumed to reach the end of the irrigation run in 1/4 of the time required for the desired depth of irrigation water to be absorbed. The water was then continued in contact until the lower end was fully irrigated. Thus, the water was in contact 5 time intervals at the upper end and 4 time intervals at the lower end. For the case considered, the exponent n in the intake equation $I = K t^n$ was assumed to have a value of -1/2 and the computed deep percolation loss was 5.3%.

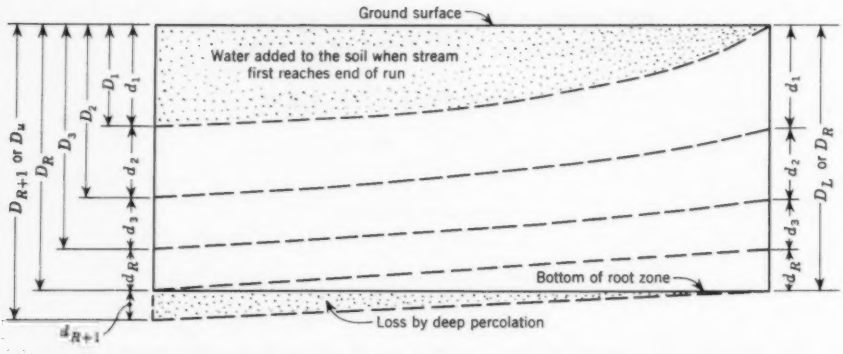

FIG. 1

Using the basic intake equation $I = K t^n$, Criddle showed that by integration an expression for depth of water absorbed in a given time would be of the form

$$d = \frac{K t^{n+1}}{60 (n+1)} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

From Eq. 1 the following analysis can be made. In Fig. 1 let:

t_1 = time required for the water to reach the end of the run;

t_2, t_3, t_4 = additional time intervals that are multiples of t_1, $t_2 = 2\,t_1$, $t_3 = 3\,t_1$, and so forth;

$t_R = R\,t_1$ = time required for the root zone to become fully irrigated with no deep percolation loss;

$R = \frac{t_R}{t_1}$ = the ratio of the time required to fill the root zone to the time required for the water to reach the end of the run;

$D_1 = d_1$ = depth absorbed in time t_1;

D_2 = depth absorbed in the time interval of $2\,t_1$;

$d_2 = D_2 - D_1$ = depth absorbed in second time interval;

3 "Methods for Evaluating Irrigation Systems," by Wayne D. Criddle, Sterling Davis, Claude H. Pair, and Dell G. Shockley, Agricultural Handbook No. 82, Soil Conservation Service, U. S. Dept. of Agric., Washington, D. C., 1956.

D_R = depth absorbed in the time intervale $t_R = R\,t_1$;

D_{R+1} = depth absorbed in the time interval $(R+1)\,t_1$;

D_L = depth absorbed at the lower end of the run = D_R if the contact time is just sufficient for the desired quantity of water to be absorbed;

D_U = depth absorbed at the upper end of the run = D_{R+1};

n = exponent of t in the intake equation $I = K\,t^n$. This is the slope of the intake line when plotted on log log paper, a minus value;

P = deep percolation loss expressed as a percentage; and

L = length of irrigation run.

From Eq. 1 it follows that

$$D_2 = \frac{K\,t_2^{n+1}}{60\,(n+1)} = \frac{K}{60\,(n+1)}\,(2\,t_1)^{n+1} = d_1(2)^{n+1} \quad \ldots\ldots\ldots (2)$$

$$d_2 = D_2 - D_1 = d_1\left(2^{n+1} - 1\right) \quad \ldots\ldots\ldots\ldots\ldots (3)$$

$$D_3 = \frac{K\,t_3^{n+1}}{60\,(n+1)} \quad \frac{K}{60\,(n+1)}\,(3\,t_1)^{n+1} = d_1(3)^{n+1} \quad \ldots\ldots\ldots (4)$$

$$d_3 = D_3 - D_2 = d_1\left(3^{n+1} - 2^{n+1}\right) \quad \ldots\ldots\ldots\ldots (5)$$

$$D_R = \frac{K}{60\,(n+1)}\,(R\,t_1)^{n+1} = R^{n+1}\,d_1 = D_L \quad \ldots\ldots\ldots (6)$$

and

$$D_{R+1} = (R+1)^{n+1}\,d_1 = D_U \quad \ldots\ldots\ldots\ldots\ldots (7)$$

As t increases the intake rate approaches a constant value that will result in a triangular pattern for the deep percolation loss below the root zone as shown in Fig. 1. The quantity of water lost per unit width will therefore be

$$V_l = (D_U - D_L)\,\frac{L}{2} = \left[(R+1)^{n+1} - R^{n+1}\right]\,d_1\,\frac{L}{2} \quad \ldots\ldots\ldots (8)$$

The total volume absorbed by the soil will be

$$V_t = \left[(R+1)^{n+1} + R^{n+1}\right]\,d_1\,\frac{L}{2} \quad \ldots\ldots\ldots\ldots\ldots (9)$$

The percentage lost will therefore be

$$P = \frac{(R+1)^{n+1} - R^{n+1}}{(R+1)^{n+1} + R^{n+1}}\,(100) \quad \ldots\ldots\ldots\ldots (10)$$

An examination of Eq. 10 shows that when n = -1, the loss will be zero for all values of R, and when n = 0 the percentage loss will be $1/(2R + 1)\,(100)$.

If it is assumed that a straight line relationship exists for values of P between n = 0 and n = -1, for any given value of R, the resulting equation for percentage water lost will be

$$P = \frac{1 + n}{2R + 1} (100) \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

The fact that no appreciable error is introduced by assuming that a straight line relationship exists in the interval $0 > n > -1$ is shown by Table 1. Col. 3

TABLE 1.—COMPARISON OF TWO METHODS OF COMPUTING DEEP PERCOLATION LOSS RATES[a]

R	n	P_1[b]	P_2[c]	Error, $P_1 - P_2$	Percentage error, $\frac{(P_1 - P_2)}{P_2} 100$
(1)	(2)	(3)	(4)	(5)	(6)
1	-0.1	30.2	30.0	0.2	0.663
	-0.3	23.8	23.3	0.5	2.1
	-0.5	17.2	16.7	0.5	2.90
	-0.7	10.35	10.0	0.35	3.5
	-0.9	3.47	3.33	0.14	4.03
2	-0.1	18.1	18.0	0.1	0.553
	-0.3	14.2	14.0	0.20	1.41
	-0.5	10.15	10.0	0.15	1.48
	-0.7	6.35	6.0	0.35	3.38
	-0.9	2.01	2.0	0.01	0.96
3	-0.1	12.8	12.82	-0.02	-0.156
	-0.3	10.0	10.0	. . .	. . .
	-0.5	7.12	7.15	-0.03	-0.421
	-0.7	4.27	4.28	-0.01	-0.234
	-0.9	1.47	1.43	0.03	2.05
4	0.1	9.97	10.0	-0.03	-0.305
	0.3	7.7	7.77	-0.07	-0.91
	0.5	5.57	5.57	. . .	. . .
	0.7	3.36	3.33	0.03	0.893
5	0.1	8.1	1.17	-0.07	-0.864
	0.3	6.38	6.36	0.02	0.314
	0.5	4.47	4.55	-0.08	-1.79
	0.7	2.7	2.73	-0.03	-1.11

[a] For various intake rates (Values of n in $I = K t^n$) and various lengths of run as a relationship to intake time in the equation $R = T_r/T_1$.
[b] From Eq. 10.
[c] From Eq. 11.

(values of P_1) shows the percentage loss as computed by Eq. 10 and Col. 4 (values of P_2) shows the percentage loss as computed by Eq. 11. The magnitude of the difference e is shown in Col. 5 and the percentage error is shown in Col. 6. The maximum error is less than 4%. Because n cannot be determined with this degree of accuracy, the use of Eq. 11 is justified to estimate deep percolation losses below the root zone for various lengths of run as determined by R. The value of n in Eq. 1 must be obtained from intake measure-

ments. Fig. 2 is a nomograph for the solution of Eq. 11. The percentage loss has been plotted against the values of n for different values of R between R = $\frac{1}{2}$ to R = 10.

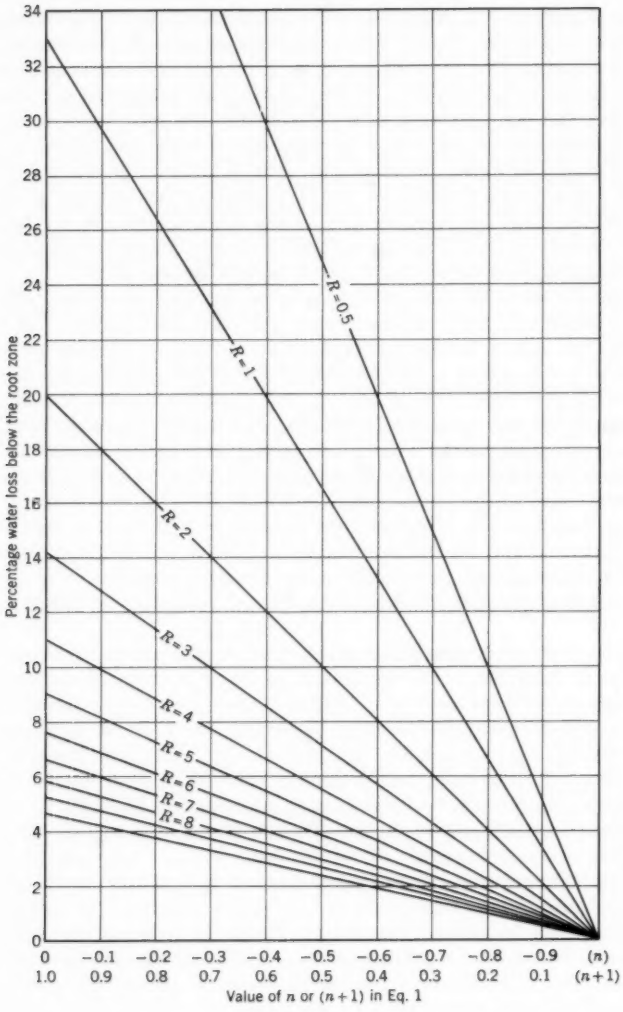

FIG. 2

An examination of Fig. 2 shows that for soils having high intake rates (n → 0) the deep percolation loss will be high if the value of R is small. On the other

hand, if the intake rate is low ($n \rightarrow -1$) the deep percolation losses will be small even for R values of 1.

The relationship expressed by Eq. 11 and shown in Fig. 2 gives the design engineer considerably more latitude in selection of the length of run to be used and allows a comparison between losses caused through deep percolation and those caused by surface runoff at the end of the run. From the irrigator's point of view, the length of run should be as long as possible, thus reducing the number of settings and also reducing the number of cross ditches and other necessary structures. The percentage of surface runoff is also reduced by having long runs (small values of R). In the same soil, assuming the water is kept at the end of the run just long enough to allow the root zone to become completely replenished, the amount of surface runoff will vary directly as R. That is, if the irrigation layout is designed with R = 3, the surface runoff will be three times what it would be with R = 1. If the surface runoff can be collected and reused this does not represent a loss to the farmer. On the other hand, if the runoff is lost to the farm the irrigation might be more efficient from the farmer's point of view with longer runs (small values of R).

This can be seen by considering a case similar to the one discussed by Criddle et al[3] in which R = 4 was suggested. In this case the n value was determined to be $-\frac{1}{2}$ and the deep percolation loss was computed to be 5.3% with runoff at the lower end of the field occurring for 80% of the time the water was being used. If R = 2 had been selected, Fig. 2 indicates that the deep percolation loss would have increased from 5.3% to 10%, an increase of 4.7%. However, the runoff time would be reduced to one half the value for R = 4. If it is assumed that the size of the runoff stream at the lower end of the field is only 10% of the stream at the upper end, the runoff loss for R = 4 would be 8% and for R = 2 it would be 4%. The net total loss (deep percolation loss plus surface runoff) would be about the same. The advantage of the longer run is the elimination of additional structures, cross ditches and additional settings, thus reducing the cost of the irrigation investment and reducing the labor required for irrigation.

The magnitude of n also plays an important role as can be observed from Fig. 2. If n is less than -0.7, the loss from deep percolation would not exceed 10% for values of R as low as 1. The fact that n is highly variable has been shown by many investigators. Data reported by G. R. Free et al[4] indicates values of n from -0.19 to -0.85. For the n = -0.19 value it would be necessary to select R = 7 if deep percolation losses were to be kept below 6%, whereas R = 1 could be used for n = -0.85 without exceeding a 6% loss.

The deep percolation loss as indicated in Fig. 2 is based on the assumption that the water is in contact at the end of the run just long enough to bring the root zone to field capacity. The losses caused by surface runoff and additional deep percolation losses that occur if the water is kept in contact longer than necessary are not included. The deep percolation loss can be kept to any reasonable value by proper selection of R as indicated by Fig. 2. Care in irrigation with regard to total time and surface runoff are perhaps even more important in the overall efficiency of irrigation.

From the practical point of view, the design of a surface irrigation layout will of necessity be a compromise between low deep percolation losses on the one hand and reduced investment and operation costs on the other, keeping in

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[4] "Relative Infiltration and Related Physical Characteristics of Certain Soils," by G. R. Free, G. M. Browning, and G. W. Musgrave, Technical Bulletin No. 729, Dept. of Agric., Washington, D. C., July, 1940.

mind the other practical and physical considerations such as surface runoff, erosion, size of farm, quality and quantity of water supply, leaching requirements to maintain a salt balance, and other factors peculiar to the farm or the area. The information herein presents a useful tool for the design engineers use in determining length of run ratio (R) when the value of n of the intake equation is known for the soil to be irrigated.

Additional information pertaining to this subject has been presented elsewhere.[5,6,7]

CONCLUSIONS

The ratio of the time required to fill the root zone to the time required for the water to reach the end of the run, R, may be varied over a wide range, depending upon the intake characteristics of the soil, the losses allowable and practical considerations. The losses by deep percolation below the root zone for various R values may be easily estimated by Eq. 11 or by Fig. 2. A useful tool for designing the length of run in a system layout of an irrigated farm is therefore available to the engineer.

ACKNOWLEDGMENTS

The writer wishes to express his appreciation to the engineers of the Aichi Irrigation Public Corporation in Japan for their assistance in developing the ideas of this paper and to Jeswant Singh, a graduate student at Utah State University, for further development of the information and the preparation of Table 1.

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5 "Report to the Government of Japan on the Irrigation of Sloping Lands," by A. Alvin Bishop, FAO Report No. 616, Rome, Italy, 1958.

6 "Report to the Government of Japan on the Irrigation of Sloping Lands," by A. Alvin Bishop, FAO Report No. 959, Rome, Italy, 1958.

7 "The Intake Rate as Related to the Advance of Water in Surface Irrigation," by J. E. Christiansen, A. Alvin Bishop, and Yu Si Fok. Presented at the December 17, 1959 Annual Meeting of the Amer. Soc. of Agric. Engrs. in Chicago, Ill.

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IRRIGATION AND DRAINAGE DIVISION
Proceedings of the American Society of Civil Engineers

VARIABLE HEAD TECHNIQUE FOR SEEPAGE METERS[a]

By Herman Bouwer[1]

SYNOPSIS

A technique for measuring canal or reservoir seepage losses with seepage meters is proposed. The method also furnishes information regarding hydraulic conductivity of bottom material and seepage gradients. A resistance network analog was used for analyses of pertinent factors and flow components, and for obtaining examples to illustrate the method.

INTRODUCTION

The bell-type seepage meter[2,3,4] offers a convenient method for measuring and localizing canal seepage losses. The general principle of determining seepage with the bell-type meter has previously consisted of duplicating outside head conditions inside the meter and determining the flow from the seep-

Note.—Discussion open until August 1, 1961. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. IR 1, March, 1961.

[a] Contribution from the Soil and Water Conservation Research Div., Agric. Research Service, U. S. Dept. Agric.

[1] Agric. Engr., Southwest Water Conservation Lab., Western Soil and Water Mgt. Research Branch, Soil and Water Conservation Research Div., Agric. Research Service, U. S. Dept. Agric., Tempe, Ariz.

[2] "Measuring Seepage from Irrigation Canals," by W. Rasmussen and C. W. Lauritzen, Agricultural Engineering, Vol. 34, pp. 326-330.

[3] "Seepage Losses from Irrigation Channels," by Carl Rohwer and Oscar Van Pelt Stout, Tech. Bulletin 38, Colorado Agric. Experiment Sta. and Soil Conservation Service, U. S. Dept. Agric., 1948.

[4] "Measuring Seepage from Irrigation Channels," by A. R. Robinson and Carl Rohwer, Tech. Bulletin 1203, U. S. Dept. Agric., 1959, p. 82.

age meter as a direct measure of the seepage from the area enclosed by the cylinder. Several techniques and apparatus have been used, as for instance the submerged flexible bag of the meter developed[4] by the Salinity Laboratory of the Dept. of Agriculture, the well and hook gage[4] of the Soil Conservation Service meter, the Mariotte syphon of the Weber Basin meter,[5] and the syphon with the calibrated glass tube through which an air bubble is passed, which is used with the Nevada seepage meter.[6]

For some of the techniques, small differences between the head inside the meter and the free water surface in the canal are difficult to avoid. The resulting exchange of water between the channel and the interior of the meter through the bottom material of the canal, may seriously affect the accuracy of the results.

Seepage meters are simple and convenient devices for measuring seepage losses, but it is recognized that refinements in the technique are needed to improve the reliability of the results.[4,7,8] A procedure is described whereby seepage is determined from deliberately created head differences between the seepage meter interior and the surrounding water, and from the corresponding flow rates into or out of the seepage meter.

Notation.—The letter symbols adopted for use in this paper are defined where they first appear, in the illustrations or in the text, and are arranged alphabetically, for convenience of reference, in the Appendix.

PRINCIPLES OF PROPOSED METHOD

The hydraulic gradients under which prolonged seepage flow takes place are made up of pressure (water depth) and elevation differences between the canal bottom and the groundwater table or some other surface of lower potential at a certain distance under the canal. On the other hand, water flow or leakage between the water inside and outside the meter is entirely a pressure flow. Consequently, pressure changes inside the meter that are small in relation to the pressure and elevation differences controlling the seepage, will practically not affect the seepage, but will cause a linear change in the leakage. (The effect of pressure as such on seepage should not be confused with that of the water depth, which, through its additional effect on wetted perimeter and geometry of the flow system, could well affect the magnitude of seepage.) This difference between seepage and leakage components in their reaction to pressure changes inside the meter is the principle of the method advanced herein. In the subsequent treatment, the assumption is made that seepage is not appreciably affected by head changes.

If the heads inside and outside the seepage meter are equal, leakage is not present and, assuming no disturbance by installation of the meter, the flow system is that of the original seepage flow prior to installation of the meter. If the head inside the meter is lowered, leakage flow begins to occur and the

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5 "Description of the Weber Basin Seepage Meter," by L. E. Myers, Bur. of Reclamation, Mimeographed Report, 1950.

6 "The Nevada Seepage Meter," by L. E. Myers, Personal Communication, 1960.

7 "Seepage Measurements-Lower Cost Canal Lining Program," Hydr. Report No. Hyd. Report No. Hyd.-297, North Platte Project, U. S. Bur. of Reclamation, Denver, Colo., 1951.

8 "Methods of Measuring Seepage Loss in Irrigation Canals," by C. C. Warnick, Bulletin No. 8, Engrg. Experiment Sta., Univ. of Idaho, Moscow, Idaho.

net flow leaving the meter through the bottom area enclosed by it becomes the resultant of the individual components Q_s, due to seepage, and Q_L due to "leakage." The resultant net flow Q_m is now the original seepage Q_s reduced by the leakage inflow Q_L. In addition lowering the head inside the meter leaves the component Q_s unaffected but increases the component Q_L, so that the resultant flow Q_m leaving the seepage meter through the channel bottom diminishes. At a certain head Q_m reaches zero $(Q_L = Q_s)$ and below this head it becomes "negative" (water entering meter through bottom area, $Q_L > Q_s$). Examples of flow systems at various head conditions inside the meter will be presented.

Without external removal or supply of water to the seepage meter, the head inside the seepage meter will establish itself at a level where inflow and outflow are in balance. The distance of this head below the water surface is called the balanced-flow differential head H_b. This term is preferred to the designation "elevation of zero seep" as given[3] by Carl Rohwer and Oscar Van Pelt because seepage is still present, but it is compensated by leakage.

The balanced-flow condition can be expressed in formula as

$$Q'_L = Q_s \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

and

$$Q_m = Q'_L - Q_s = 0 \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which

$$Q_s = \pi \, R_c^2 \, I_s \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

and Q'_L is the leakage flow at H_b. In Eq. 3 R_c is the radius of the seepage meter cup and I_s denotes the seepage rate. The term H_b denotes the balanced flow differential head. Reducing the head inside the meter to a level of 2 H_b will double the leakage. With the assumption that the seepage component remains unaffected, the net flow that must now be removed externally from the meter is

$$Q_m = 2 \, Q'_L - Q_s \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (4)$$

In general, a head difference of $H = n \, H_b$ will yield

$$Q_m = n \, Q'_L - Q_s \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (5)$$

Substituting Eq. 1 into Eq. 5 gives

$$Q_m = (n-1) \, Q_s \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (6)$$

from which the seepage rate I_s can be computed as

$$I_s = \frac{Q_m}{(n-1) \, \pi \, R_c^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7a)$$

It can be seen from Eq. 7a that a minimum of two observations are necessary to compute I_s; that is H_b in which $Q_m = 0$ and another H-value with corresponding Q_m, $(n = H/H_b)$. It is actually proposed to make a series of observations to obtain a statistically more reliable estimate of I_s. Taking Q_m-measurements for various H-values, Q_m can be plotted against n-1. According to Eq. 7a this plot should be linear and pass through the origin. The rate I_s can then be computed as

$$I_s = \frac{a}{\pi \, R_c^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (7b)$$

in which "a" is the slope of the best-fitting line in the Q_m versus n-1 plot.

If saturated conditions exist in the region in which most of the leakage flow takes place, Q_m will respond rapidly to changes in H, which suggests the possibility of using variable head techniques. Such techniques are easier to carry out than constant-head procedures. If the measurements are taken in a container connected to the seepage meter, the rate of rise in the container can be expressed according to Eq. 7 as

$$\frac{dH}{dt} = -(n-1) I_s \frac{R_c^2}{R_v^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Eq. 8 can be integrated to

$$I_s = \frac{2.3 H_b R_v^2}{t R_c^2} \log \frac{H_o - H_b}{H_t - H_b} \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which H_o = H at t = 0, and H_t = H at time t. The term R_v refers to the radius of the vessel in which the rate of change of pressure head in the seepage meter is measured.

Taking readings of H at various t-values enables plotting of $(H_o-H_b)/(H_t-H_b)$ against t on semilogarithmic paper, after which the slope of the best-fitting straight line is used to compute I_s according to Eq. 9. By proper selection of R_v, it should be possible to complete the necessary measurements in a relatively short period of time. Rising or falling water levels, or combinations of them, may be used for obtaining the necessary H-t measurements.

ELECTRICAL RESISTANCE NETWORK STUDIES

Quantitative aspects of leakage flow and head conditions at balanced flow were carried out[9] with an electrical resistance network (Fig. 1). The easy control of boundary, geometry, and conductivity conditions with such an analog makes it a convenient and superior device for analyzing a variety of flow systems. The analog was also used to illustrate the method of seepage measurement with some examples, and to determine the effect of nonuniform soil conductivity and nonuniform seepage rates on the seepage meter performance.

The dimensions of the flow system represented on the analog were selected as 8.48 R_c in diameter and 3.43 R_c in depth. At these dimensions, increasing the diameter and depth of the flow system to infinity had a negligible effect on the measured flow rates (less than 0.65% for the leakage rates given in Fig. 2). Thus the system on the analog was large enough for adequate representation of the "open" flow system as it occurs under actual canals.

The density of the network was greatest in the region near and under the rim of the seepage meter (Figs. 5 through 8). The resistances for this subdivided radial arrangement were computed similarly to a procedure presented previously.[10] The seepage meter penetration was simulated by using a double line of resistors which were computed as an inner and outer solid cylindrical boundary, respectively.

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[9] "A Unifying Numerical Solution for Two-Dimensional Steady Flow Problems in Porous Media with an Electrical Resistance Network," by H. Rouwer, Proceedings, Soil Science Soc. of America, Vol. 23, 1959, pp. 91-96.

[10] "A Study of Final Infiltration Rates from Cylinder Infiltrometers and Irrigation Furrows with an Electrical Resistance Network," by H. Bouwer, Proceedings, Seventh Congress Internatl. Soc. of Soil Science, Madison, Wis., 1960.

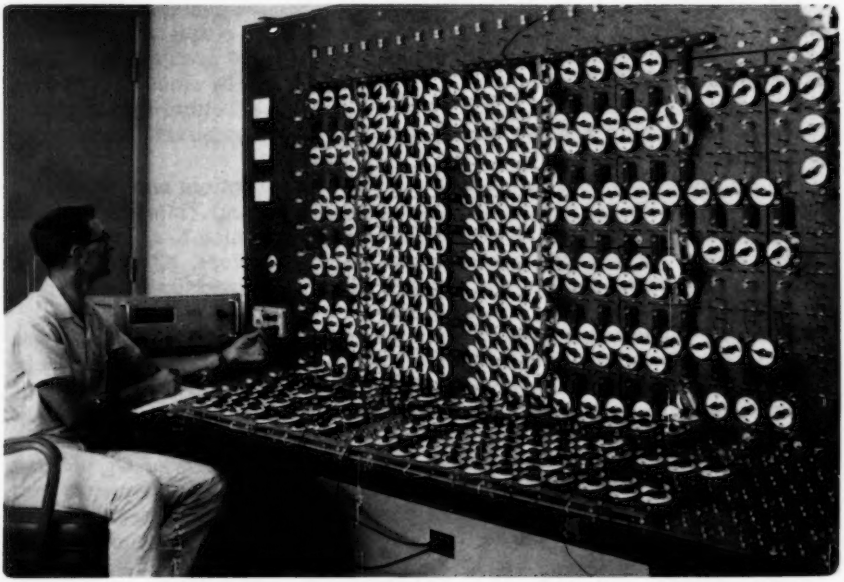

FIG. 1.—ELECTRICAL RESISTANCE NETWORK USED IN THE SEEPAGE ANALYSES

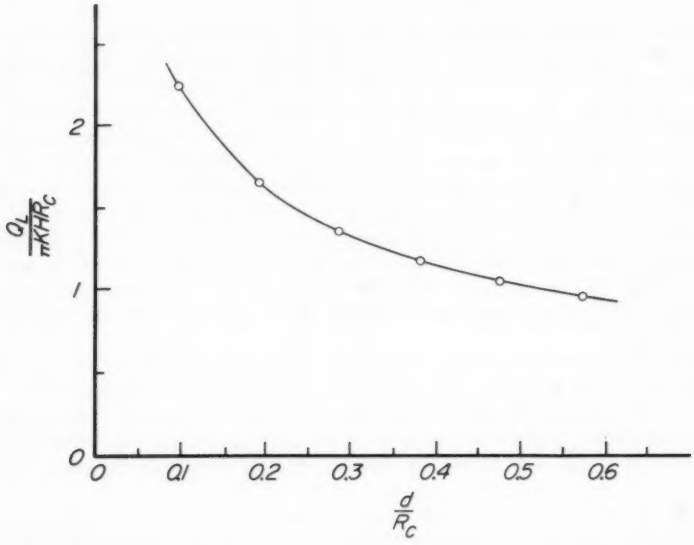

FIG. 2.—RESULTS OF THE LEAKAGE FLOW ANALYSES IN DIMENSIONLESS FORM

The equipotentials in the limited flow region under the center of the channel before installation of the meter can, at steady or near-steady conditions, be considered horizontal in the light of previous work on flow systems under partial inundations.[10,11] Thus, different seepage rates can be simulated by simply applying different voltage drops between the channel bottom (upper equipotential) and the bottom of the flow system (lower equipotential) represented on the analog.

The pressures at small distances below the channel bottom may exceed atmospheric pressure. At greater depths, tensions may occur. In uniform soils, however, these tensions will not exceed the tension at which K-reductions occur in the flow region under consideration.[10,11] Therefore, K, the hydraulic conductivity of bottom material, is equal or close to the saturated conductivity and Q_m responds rapidly to changes in H.

Leakage.—To determine the importance of the leakage flow alone, Q_L was determined as a function of d/R_c in the absence of seepage. This was done by measuring Q_L as electrical current at a certain voltage difference H. The relationship between Q_L and H is obviously linear. A dimensionless plot of the results is shown in Fig. 2. The parameter $Q_L/(\pi K H R_c)$ is obtained by expressing the rate of rise in a cylinder of R_c due to the inflow of Q_L per unit permeability, that is $Q_L/(\pi R_c^2 K)$, and dividing this term by a geometry factor describing H in relation to d, the depth of penetration of the seepage meter in the canal bottom, or R_c. Since R_c already appeared in the rate of rise per unit K, H/R_c was selected, yielding

$$\frac{\dfrac{Q_L}{\pi R_c^2 K}}{\dfrac{H}{R_c}} = \frac{Q_L}{\pi K H R_c} \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

The results indicate that leakage flow can be quite appreciable, even for small H-values, and that it may well approach the magnitude of the seepage flow if I_s is small compared to the K of the material in which most of the leakage flow takes place. For instance if R_c = 21 cm, d = 4 cm, and H = 1 cm, the inflow in the seepage meter, expressed as the average upward flow velocity in the cylinder, can be computed from Fig. 2 as 0.08 K. For large canals, uniform soils, and unrestricted disposal of mainly vertical seepage below the underground water table, I_s will approach K. Therefore, a head difference of 1 cm would result in an error of the seepage I_s of about 8%, if the technique based on duplication of outside head conditions inside the meter were used. However, if K in the region of the leakage flow exceeds the K of the region controlling the seepage, inaccuracies in head duplication may result in a much more serious error of I_s.

The same is true for restricted disposal, such as lateral flow due to an impermeable layer, for which I_s may only be a fraction of K. Leakage flow may explain C. C. Warnick's, M. ASCE, conclusion[8] that "... some of the inconsistency in the seepage meter results is due to the bag not acting as a perfect membrane to transmit the pressure equally from the outside to the inside of the bag."

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11 "Theoretical Aspects of Unsaturated Flow in Drainage and Subirrigation," by H. Bouwer, Agricultural Engineering, Vol. 40, 1959, pp. 395-400.

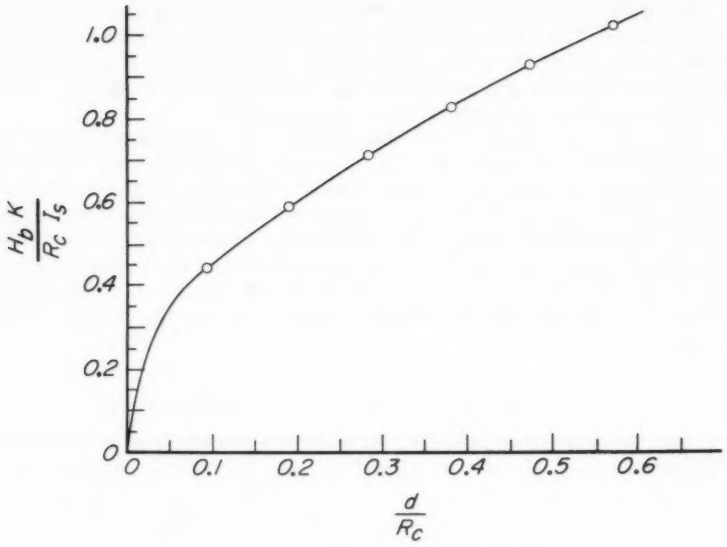

FIG. 3.—RESULTS OF THE ANALYSES OF THE BALANCED-FLOW DIFFERENTIAL HEAD IN DIMENSIONLESS FORM

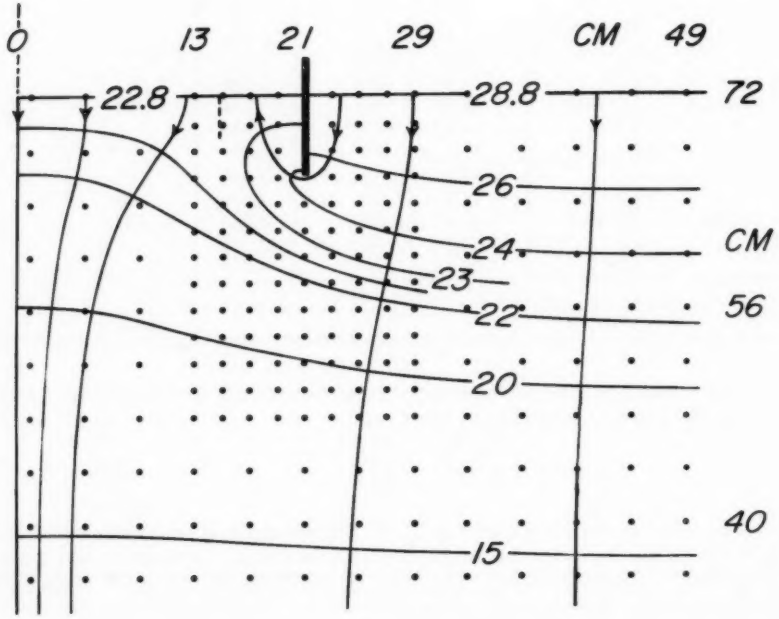

FIG. 4.—FLOW SYSTEM AT BALANCED-FLOW CONDITIONS

As could be expected, Q_L increases with decreasing depth of penetration of the cylinder. Thus, efforts to reduce soil disturbance by minimizing the depth to which the meter should be placed in the bottom of the channel[12] would at the same time increase the possibility for leakage, which would be undesirable if procedures based on duplication of head are used.

Balanced-Flow Head Differential.—The head difference H_b at balanced flow was determined in relation to d/R_c. Electrical current was only applied to the "canal bottom outside the seepage meter." The electrical potential of the conductor representing the bottom enclosed by the seepage meter was then measured. This potential is directly proportional to the voltage difference between the outside canal bottom (upper equipotential) and the bottom of the flow system (lower equipotential). This means that for a given soil and a given meter and depth of installation, the relation between H_b and the seepage gradient I_s/K is linear.

TABLE 1.—RESULTS OF SEEPAGE TESTS[a]

Actual seepage I_s/K (1)	H_b cm (2)	H cm (3)	Q_{m_2}/K cm (4)	n-1 (5)	Factor a, in Eq. 7b (6)	Seepage computed with Eq. 7b, I_s/K (7)
		1.7	17.6	0.113		
		2.5	89.2	0.667		
0.100	1.5	3.5	177	1.33	133	0.0964
		4.5	267	2.00		
		5.5	356	2.67		
		7.0	92.3	0.167		
		8.0	181	0.333		
		9.0	279	0.500		
0.400	6.0	10.0	366	0.667	554	0.400
		11.0	462	0.833		
		13.0	647	1.167		
		22.8	182	0.096		
		24.8	364	0.192		
1.39	20.8	26.8	534	0.288	1847	1.33
		28.8	710	0.385		
		34.8	1243	0.673		

[a] Electrical Resistance Network, R_c = 21 cm and d = 6 cm.

The dimensionless parameter for generalization of the results was selected as H_b/R_c per unit seepage gradient, or

$$\frac{H_b/R_c}{I_s/K} = \frac{H_b K}{R_c I_s} \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (11)$$

The results are shown in Fig. 3. If, for example, R_c = 21 cm, d = 4 cm, and I_s = 0.4 K, H_b can be computed from Fig. 3 as 6 cm. An example of a flow sys-

[12] "The Design and Testing of Equipment for Measuring Seepage Losses from Canals," by Stuart Meerscheidt, thesis presented to Utah State Univ., at Logan, in 1951, in partial fulfilment of the requirements for the degree of Master of Science.

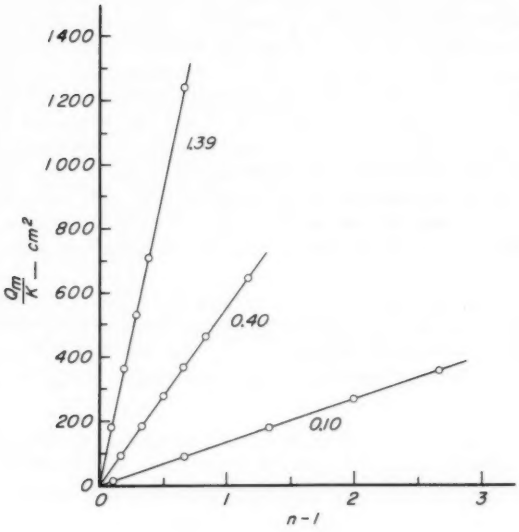

FIG. 5

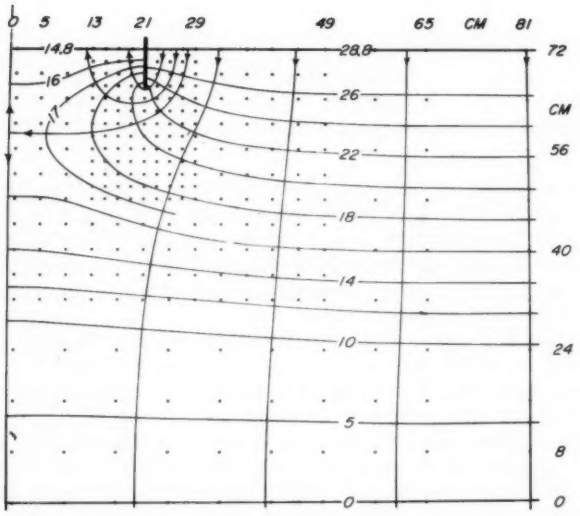

FIG. 6.—FLOW SYSTEM FOR H > H_b SHOWING COMPLETE NETWORK ARRANGEMENT USED IN ANALYSES.

tem at balanced flow is shown in Fig. 4. In Fig. 4 $I_s/K = 0.4$, $d/R_c = 0.286$ and $H_b = 6$ cm. Only the portion of the network in the vicinity of the meter is shown. The network nodes are indicated by the dots. The seepage meter is shown as a heavy line at a distance of 21 cm from the center. The equipotentials are expressed in cm and refer to a datum at 72 cm below the channel bottom. The streamlines were sketched as orthogonals to the equipotentials.

According to Eq. 1, $Q_L = Q_S$ when $H = H_b$. At balanced flow conditions, therefore, the parameter on the ordinate of Fig. 2 can be expressed as $Q_S/(\pi\ K\ R_c\ H_b)$ or, because $Q_S = I_s\ \pi\ R_c^2$, as $(I_s\ R_c)/(H_b\ K)$. This term is the reciprocal of the parameter on the ordinate of Fig. 3, so that the curve in Fig. 3 must be the reciprocal of the curve in Fig. 2. The "reciprocal" relationship between the two curves, which were obtained in different ways, appears to be excellent.

Examples of Seepage Measurement. — Seepage measurements were preformed by simulating seepage meter tests on the network analog under various conditions of known (imposed) seepage rates. The results (Table 1) show an excellent agreement between the actual seepage imposed on the systems and the

TABLE 2.—COMPARISON OF SEEPAGE OUTFLOW AND LEAKAGE INFLOW TO SEEPAGE METER AT BALANCED-FLOW LEVEL.

Actual seepage in terms of I_s/K (1)	Computed seepage component from cylinder (in sq cm) $\frac{Q_s}{K} = \frac{I_s\ \pi\ R_c^2}{K}$ (2)	Leakage component Q'_L/K into cylinder computed from figure 2, taking $H = H_b$ from Table 1 (in sq cm) (3)	$\frac{Q_s}{Q'_L}$ (4)
0.10	138	135	1.02
0.40	554	538	1.03
1.39	1921	1865	1.03

seepage rates computed with Eq. 7b from analogous Q_m and H data. The Q_m versus n-1 plots showed excellent straight-line relationships (Fig. 5). The I_s/K-values are indicated in Fig. 5, by the numbers on the lines. The small discrepancies between the imposed and measured seepages (Columns 1 and 7 in Table 1) must be attributed to the overall accuracy of the analog instrumentation, which is approximately 3%.

The equality of leakage and seepage components at balanced flow appears from Table 2, in which, for the three I_s/K-values of Table 1, the leakage for H_b computed from Fig. 2 is in agreement with the seepage as computed from the actual seepage rates imposed on the system.

Examples of flow nets are given for a case of balanced-flow (Fig. 4) and a case of $H > H_b$ with $I_s/K = 0.4$, $d/R_c = 0.286$ and $H = 14$ cm (Fig. 6). Although the region affected by the seepage meter flow extends beyond the area enclosed by the cylinder (Fig. 7), the measured seepage applies to the cylinder area only. In Fig. 7 normal seepage flow is represented by $H = 0$, balanced flow by $H = 6$ cm, and two values of H in excess of H_b by $H = 10$ cm and $H = 14$ cm. For all values of H, $I_s/K = 0.4$ and $d/R_c = 0.286$. This was demonstrated with the analog by raising the portion of the lower equipotential under the seepage

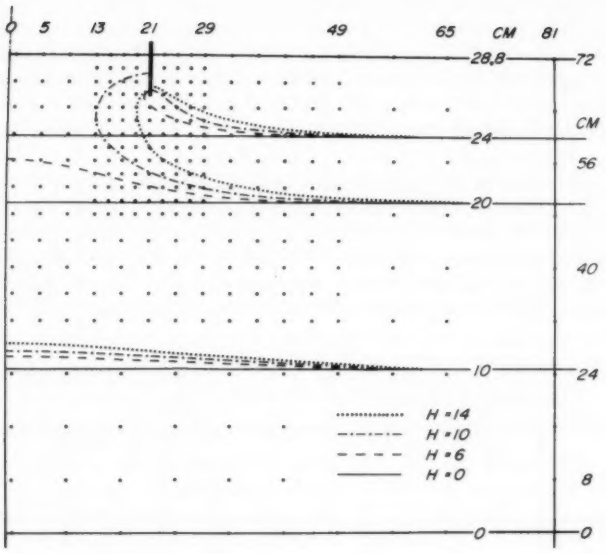

FIG. 7.—SELECTED EQUIPOTENTIALS IN CENT METERS FOR FLOW SYSTEM AT VARYING FLOWS.

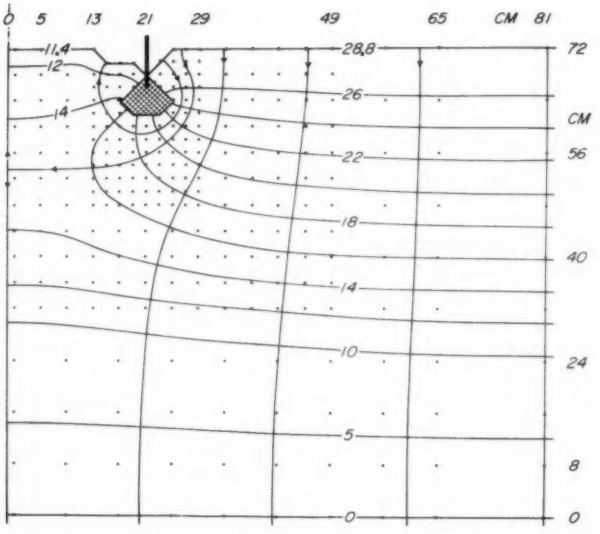

FIG. 8.—FLOW SYSTEM FOR $H > H_b$ WITH SOIL DISTURBANCE NEAR SEEPAGE METER.

meter a certain distance, which caused the average seepage, at the location of the meter, prior to installation, to be 1.3 times the average seepage outside the meter; that is, 0.59 K and 0.46 K, respectively. Simulating a seepage test on the analog similar to the tests listed in Table 1 yielded a value of 0.57 K, or the seepage as it occurred in the area enclosed by the meter before the meter was installed. Other tests with non-uniform seepage rates gave similar results.

Non-Uniformity and Disturbance of Bottom Material.—Since the method presented in this paper is based on the linearity between H and Q_L, its applicability should not be limited to uniform soil conditions. This was evidenced by an analog seepage test for which the conductivity of the top 8 cm of the medium was four times the conductivity of the subsoil. The measured seepage was 0.403 $K_{subsoil}$, which agreed with the imposed seepage of 0.400 $K_{subsoil}$.

TABLE 3.—RESULTS OF SEEPAGE TESTS WITH RESISTANCE NETWORK ANALOG FOR SIMULATED SOIL DISTURBANCE NEAR SEEPAGE METER[a]

(1)	I_s/K (2)	H_b in cm (3)	H in cm (4)	Q_m/K sq cm (5)	n-1 (6)	a (7)	I_s/K (8)
Combined effects of loosening at surface and compaction below rim	0.40	9.4	11.4 13.4 15.4 17.4 19.4 21.4	108 217 325 433 540 648	0.213 0.426 0.639 0.851 1.064 1.280	508	0.37
Loosening of surface soil only	0.40	4.7	-	-	-	-	0.44
Compaction below rim only	0.40	9.5	-	-	-	-	0.35

[a] As shown in Fig. 8 (columns identical to Table 1)

Local soil disturbances due to placement of the seepage meter should not be of importance either, provided that the overall seepage pattern at some distance under the meter is not affected and that linearity between H and Q_L is maintained. The latter implies that the meter should be installed so as to avoid free passages which would cause the leakage flow to be no longer governed by the Darcy equation.

The single as well as the combined effects of a loosening of the surface soil inside and outside the meter and a compaction of the soil below the rim of the cylinder due to installation were evaluated with the analog. The disturbance was simulated by shorting (infinite K) and removing (zero K) a number of resistors as shown in Fig. 8. In Fig. 8 zero permeability applies to the hatch section and infinite permeability for the surface soil near the rim of the meter as indicated ($I_s/K = 0.4$, $d/R_c = 0.286$ and $H = 17.4$ cm). The results indicate that the single and combined effects of the disturbances are small (Table 3).

Plotting Q_m/K against n-1 for the combined-effects test yielded a straight line that was close to the $I_s/K = 0.40$ line in Fig. 5. Straight-line relationships were also obtained for the other two tests of Table 3. An example of a flow system with loosened surface soil and compacted soil under the meter is shown in Fig. 8.

Since the disturbance should not be of such an extent that the seepage pattern is appreciably affected, extreme care should always be exercised to install the meter with a minimum of soil disturbance. This is particularly important if the seepage is controlled by a relatively thin layer of slowly permeable material at the surface of the channel bottom.

Computation of Hydraulic Conductivity.--The procedure in this paper also furnishes information regarding the conductivity in the region at which the seepage flow is measured. For nonuniform media, this would be an average or equivalent value; specifically, it would be the K of a uniform medium that, under the same flow conditions, would yield the same seepage as the actual medium of nonuniform K. After computation of I_s, the conductivity can be determined from Fig. 3, for which K is the only unknown in the parameter $(H_b\ K)/(R_c\ I_s)$. The seepage gradient can then be computed as I_s/K.

Fig. 3 applies to a medium that is uniform to a depth of at least 3.4 R_c. Another paper will present the determination of hydraulic conductivity from leakage flow measurements when the medium is not uniform to that depth, but is underlain at shallower depth by material of much higher or much lower conductivity than the bottom material.[13]

CONCLUSIONS

The paper is based on the assumption that seepage is not affected by head changes. This assumption will be true for seepage rates that are numerically less than the conductivity of the bottom material. The assumption does not hold if the bottom material is of relatively low conductivity and underlain at shallow depth by a well-drained soil of higher conductivity. In that case, the seepage tends to numerically exceed the conductivity of the bottom material. The writer plans to show at some future date how the principles of the present paper can be developed into a suitable field method for seepage and conductivity measurement, and how the method can be applied to conditions of head-affected seepage. It will be shown that, in the latter case, information is not only obtained regarding seepage and conductivity of bottom material, but also regarding the thickness of the slowly permeable material controlling the seepage.

In previous methods of measuring water losses with seepage meters, investigators generally attempted to maintain the head in the seepage meter equal to the head in the canal so that the meter outflow was a direct measure of seepage. Equal heads, however, are not always easy to maintain and it has been shown that even small head differences may cause appreciable error in the results. In other methods for measuring seepage or conductivity with variable head techniques, the assumption was made that the length of the soil column in which head differences were dissipated was equal to the depth of penetration of the meter.[3,4] This assumption is only valid if the bottom is underlain by well-drained more permeable material to which the meter penetrates.

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13 "A Double-Tube Method for Measuring Hydraulic Conductivity of Soil in Situ Above a Water Table," by Herman Bouwer, submitted for publication in Proceedings, Soil Science Society of America, 1961.

The manner in which seepage can be determined by allowing the head inside the seepage meter to vary while simultaneously measuring the meter inflow or outflow has been demonstrated. From electrical analogy studies, these observations also yield information regarding hydraulic conductivity of bottom material and seepage gradients. The field technique consists essentially of measuring rate of change of water level in a container connected to the seepage meter.

Examples of seepage measurements, obtained with a resistance network analog, showed excellent agreement between imposed and measured seepage rates. The analog also showed that the procedure in this paper is applicable to nonuniform conditions of soil or unequal distribution of seepage rates.

APPENDIX.—NOTATION

d = depth of penetration of seepage meter in canal bottom;

H = distance of pressure head in seepage meter below (positive) or above (negative) water surface in canal;

H_b = balanced-flow differential head (H where seepage outflow and leakage inflow are in balance, $Q_m = 0$);

I_s = seepage rate (dimensions L/T);

K = hydraulic conductivity of bottom material (L/T);

Q_s = seepage meter flow component due to seepage ("seepage" flow, $Q_s = \pi R_c^2 I_s$);

Q_L = seepage meter flow component due to flow between seepage meter and outside canal through the bed material under the meter ("leakage" flow, L^3/T);

Q_m = net volume rate of flow into or out of seepage meter that must be externally withdrawn or supplied, respectively, to maintain steady conditions ("measured" flow, L^3/T);

R_c = radius of seepage meter cup;

R_v = radius of vessel in which rate of change of pressure head in seepage meter is measured; and

t = time.

Journal of the
IRRIGATION AND DRAINAGE DIVISION
Proceedings of the American Society of Civil Engineers

DISCUSSION

Note.—This paper is a part of the copyrighted Journal of the Irrigation and Drainage Division, Proceedings of the American Society of Civil Engineers, Vol. 87, No. IR 1, March, 1961.

DRAINAGE PROBLEMS IN THE SAN JOAQUIN VALLEY[a]

Closure by William L. Berry and Edward D. Stetson

WILLIAM L. BERRY,[1] F. ASCE, and EDWARD D. STETSON,[2] M. ASCE.—The writers were quite interested in W. W. Donnan's discussion of possible alternative physical solutions to the problem of providing drainage disposal to the Tulare Lake Basin of the San Joaquin Valley.

The paper contained no specific solution to the drainage disposal problem because this naturally must await determination of the present and future drainage requirements of each area of the valley, as outlined in the paper. However, considerable progress has been made in the writers studies since presentation of the paper, and they are now studying various alternative specific solutions. An alignment skirting the Tulare Lake Bed on the east, somewhat similar to the one described by Mr. Donnan, has been given careful consideration, and may be recommended as a feature for ultimate construction.

The writers agree with Mr. Donnan that overall salt balance is important, and this was pointed out in the paper. However, in an area such as the Tulare Lake Basin in which salts have been accumulating for thousands of years, the provision of overall salt balance does not necessarily ensure the continued productivity of segments of the economy dependent on a water supply of a given quality. In our planning, consequently, we are more concerned with concentrations of salts than we are with tons of salt.

Mr. Donnan mentions the possibility of desalinization as an alternative to disposal from the basin of the waste water. This is of particular interest in connection with the treatment of wastes originating at some distance from the main waste-production areas. However, even in those circumstances, the cost does not appear to be competitive with hydraulic disposal. It should be noted that after desalinization the salt still requires disposal from the basin.

The writers greatly appreciated the discussion of the paper by a man of Mr. Donnan's recognized stature in the field of drainage.

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[a] September 1959, by William L. Berry and Edward D. Stetson (Proc. Paper 2160).
[1] Chf. Div. of Resources Planning, Calif. State Dept. of Water Resources.
[2] Sr. Hydr. Engr., Div. of Resources Planning, Calif. State Dept. of Water Resources.

METHODS OF APPLYING IRRIGATION WATER[a]

Discussion by Gregory Efstratiadis

GREGORY D. EFSTRATIADIS,[2] M. ASCE.—The writer will comment on one point that might strengthen prejudices against demand systems. Reference is made to the author's statement that "It is readily apparent that a 'demand' system.....would dictate much greater capacity than a 'rotation' system..."

It is assumed that the author means by rotation system any delivery system in which all available irrigation outlets have to be in continuous operation during the whole of the available time of the period of peak requirements of crops. A rotation system thus defined may actually work either on a fixed schedule of water rotation or without such a schedule. The schedule may be avoided, or more correctly become superfluous, if each individual farm is provided with an irrigation outlet having a capacity that is just enough to take care of peak requirements provided it operates continuously.

The demand system should then be understood as a delivery system in which there are facilities of providing water at such a rate that the available irrigation outlets do not have to operate during the whole period of peak requirements of crops but only during a fraction of that time. A demand system may also work on a schedule less rigid and usually involving a smaller number of farms per irrigation outlet, or it may work without one. As in the previous case, the schedule could be avoided if each individual farm possesses its own outlet, and if the capacity of the outlet corresponds exactly to the acrage of the farm.

It may be seen from the foregoing that in as far as conduit capacity is concerned, the distinction between rotation and demand is rather irrelevant and there is another feature of the delivery system that determines what capacities should be selected for the various conduits. This feature has been described as "probability of operation" of each irrigation outlet.[3] This probability is defined as the ratio of the time required by an irrigation outlet to pass the quantity of water necessary to the crops of the corresponding area (during the period of peak requirements), divided by the total useful time period of peak requirements. An irrigation system with a probability of operation of 1:1 is evidently the one requiring minimum conduit capacities. An irrigation system with probabilities below 1:1, for example, 1:2 or 1:3, obviously requires greater installed capacities.

A demand system being necessarily a system with probability of operation less than 1:1, the author's statement to the effect that a demand system dic-

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[a] September 1960, by Paul H. Berg (Proc. Paper 2595).

[2] Cons. Engr., Athens, Greece.

[3] "Le calcul des débits dans les canalisations d'irrigation," by René Clément, Ass. Amicale des Anciens Elèves de l'Ecole Nationale du Génie Rural, Paris, 1956.

tates greater capacities is perfectly correct. However, the adjective greater should be qualified to avoid misunderstandings.

Appreciably greater capacities are actually required in the case of probabilities much lower than 1:1, and even then only in conduits serving a limited number of irrigation outlets. The capacity of main conduits serving a few hundred outlets is, for all practical purposes, scarcely affected by the probability of operation. René Clément has established a very simple formula[3] by which the required capacity at the upstream end of a delivery system may be computed. This formula is based on simple probability concepts, takes account of the total number of available irrigation outlets and of the probability of operation, and gives a value for the capacity at the upstream end that covers any desired percentage of all possible combinations of open and closed irrigation outlets. Clément's formula may be applied to any point of the delivery system in addition to the upstream end, provided that a correction of said percentage is introduced. A method of computing this correction has been proposed by the writer.[4] The importance of Clément's formula may be illustrated by means of a numerical example.

Example.—An irrigation system contains 300 outlets. The capacity of each outlet is three times the one required to pass the necessary water quantity under continuous operation. This means that the probability of operation in the system is 1:3. Then, a capacity at the upstream end only 13.5% greater than the corresponding capacity for the case of a probability of 1:1, would take care of as much as 95% of all possible combinations of the 300 outlets being open or closed. If the number of outlets is 900 instead of 300, the corresponding increase of the capacity would be less than 8%.

It may be concluded that with approximately the same size of main conduit, the same pumping station and the same 24 hr storage facilities we may have a delivery system with a probability of 1:3 rather than 1:1, that is, a system with much greater operational flexibility and better adapted to intensive agricultural development.

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4 "Sur le mode de calcul des réseaux d'irrigation sous pression d'après la méthode Clément," L'Irrigazione, No. 4, Verona, Italy, 1960.

ERRATA

Journal of the Irrigation and Drainage Division

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Discussion by R. William Nelson, June 1960, Proc. Paper 2536, of "Laboratory Research on Interceptor Drains" by Jack Keller and A. R. Robinson.

P. 74. In the caption for Fig. 1 change SCHEMA to SCHEME.

p. 75. Eq. 4 should read as follows:

$$\frac{\partial^2 \phi}{\partial X^2} + \frac{\partial^2 \phi}{\partial Z^2} = 0$$

p. 76. In the line following Eq. 14 change "assumed" to "assured."

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2703 is identified as 2703(ST1) which indicates that the paper is contained in the first issue of the Journal of the Structural Division during 1961.

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MARCH: 2393(IR1), 2394(IR1), 2395(IR1), 2396(IR1), 2397(IR1), 2398(IR1), 2399(IR1), 2400(IR1), 2401(IR1), 2402(IR1), 2403(IR1), 2404(IR1), 2405(IR1), 2406(IR1), 2407(SA2), 2408(SA2), 2409(HY3), 2410(ST3), 2411(SA2), 2412(HW1), 2413(WW1), 2414(WW1), 2415(HY3), 2416(HW1), 2417(HW3), 2418(HW1)[c], 2419(WW1)[c], 2420(WW1), 2421(WW1), 2422(WW1), 2423(WW1), 2424(SA2), 2425(SA2)[c], 2426(HY3)[c], 2427(ST3)[c].

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c. Discussion of several papers, grouped by divisions.

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